

**Kansas Citys, Missouri and Kansas  
Flood Damage Reduction Feasibility Study  
(Section 216 – Review of Completed Civil Works Projects)  
Engineering Appendix to the Interim Feasibility Report**

## **Chapter A-7**

# **GEOTECHNICAL ANALYSIS FAIRFAX-JERSEY CREEK (BPU FLOODWALL)**

**CHAPTER A-7**  
**GEOTECHNICAL ANALYSIS**  
**FAIRFAX-JERSEY CREEK (BPU FLOODWALL)**

**A-7.1 INTRODUCTION**

This chapter presents the results of the geotechnical evaluation of an area in the Fairfax-Jersey Creek protection unit determined to have a certain probability of failure under the existing level of protection that warranted further study. The determination relies on historical borings and soil test information combined with recent subsurface borings and soil test information.

**A-7.2 SOURCES OF EXISTING LEVEE DESIGN INFORMATION**

The primary sources of information for this geotechnical analysis include the references listed in the References section of this chapter.

**A-7.3 DESCRIPTION OF THE LEVEE UNIT**

Refer to Section A-4.3.7 for a detailed description of the Fairfax-Jersey Creek Unit.

**A-7.4 LEVEE DESIGN FEATURES**

**A-7.4.1 Existing Levee and Floodwall Sections**

The Fairfax-Jersey Creek levee unit consists of levees, floodwalls, stoplog and sandbag gaps, riprap and levee toe protection, surfaced levee crown and ramps, drainage systems, the Jersey Creek sewer structure and shutter gates, and pumping plants. It was originally constructed as a local levee, but was removed and replaced using Federal standards in 1940. The final contract for construction of the project was completed in 1955.

A plan view of the Fairfax-Jersey Creek Unit and typical sections are provided in the Supplemental Exhibits section as Exhibits A-7.1 through A-7.11.

**A-7.4.2 Future Flood Protection Concerns**

This levee unit is being considered for a partial reestablishment of existing design level of protection near the Kansas River, or the beginning of the project. Some areas have been surveyed and appear to be lower than the original federal design elevation. The remainder of the system is not recommended for a raise based on the hydraulic analysis of the Missouri and Kansas Rivers.

As a result of an evaluation of the existing floodwall near the BPU facilities, it was concluded that Stations 287+85 to 302+32 required further evaluation. The 1993 flood did not reach the top of the floodwall. A full head to the top of the floodwall is analyzed in this chapter.

**A-7.4.3 Area Site Characterization**

The Corps of Engineers' boring and additional subsurface information shown in the Supplemental Exhibits section (Exhibits A-7.12 and A-7.13) was used to characterize

and model the foundation resistance for the existing precast concrete piles driven below the existing floodwall. The boring assignment for investigations of the Fairfax area was developed well in advance of the floodwall concerns. One Corps boring was proposed earlier, but was designated for investigation of an existing stoplog gap. The boring was eliminated when it was discovered that the stoplog gap had been permanently sealed. No additional borings were assigned for the study of the floodwall during the earlier stage of general investigations. Later, when concerns were raised regarding the integrity of the floodwall pile length and strength, additional investigations were conducted through a contractor. The Investigation Report is included as Exhibit A-7.14.

These borings were used for developing the resistance to vertical and horizontal loading. The characterization of the floodwall foundation relied on the Standard penetration resistance obtained by driving a split spoon through the foundation sands. An automatic trip hammer was used and appropriate corrections were made for energy input, overburden pressures, rod type, rod length, hole size, and other pertinent variables during the test. This information was used to develop the expected soil strength of the foundation sands (refer to Exhibits A-7.17 through A-7.20).

#### **A-7.4.4 Pile Capacity**

The soil strength and existing pile length and configuration below the pile cap was used to develop a spreadsheet model for determination of the available amount of side friction and end bearing for a circular driven precast concrete pile. The lateral coefficient of friction was chosen for the driven condition and a correction to the resistance was made, taking into consideration the expected foundation underseepage pressure during a high river condition. A flow net was used to model the underseepage pressures. This information was provided to the structural engineering team members for their use. Exhibit A-7.15 and Exhibits A-7.21 through A-7.23 contain the geotechnical calculations for pile capacity. A summary of the ultimate foundation resistance is provided in Exhibit A-7.24.

#### **A-7.4.5 Underseepage Analyses**

The model used to develop the expected pressure in the foundation sands along the length of the driven piles was a simple hand-drawn flow net. The cutoff wall was ignored and equipotential lines with perpendicular flow lines were developed below a mostly sand foundation. The equipotential lines were used to determine the pressure along the three piles located landside, the middle piles, and the piles riverside of the stem of the floodwall. These tabulated values were added to the model for the resistance to correct for lower effective overburden pressure.

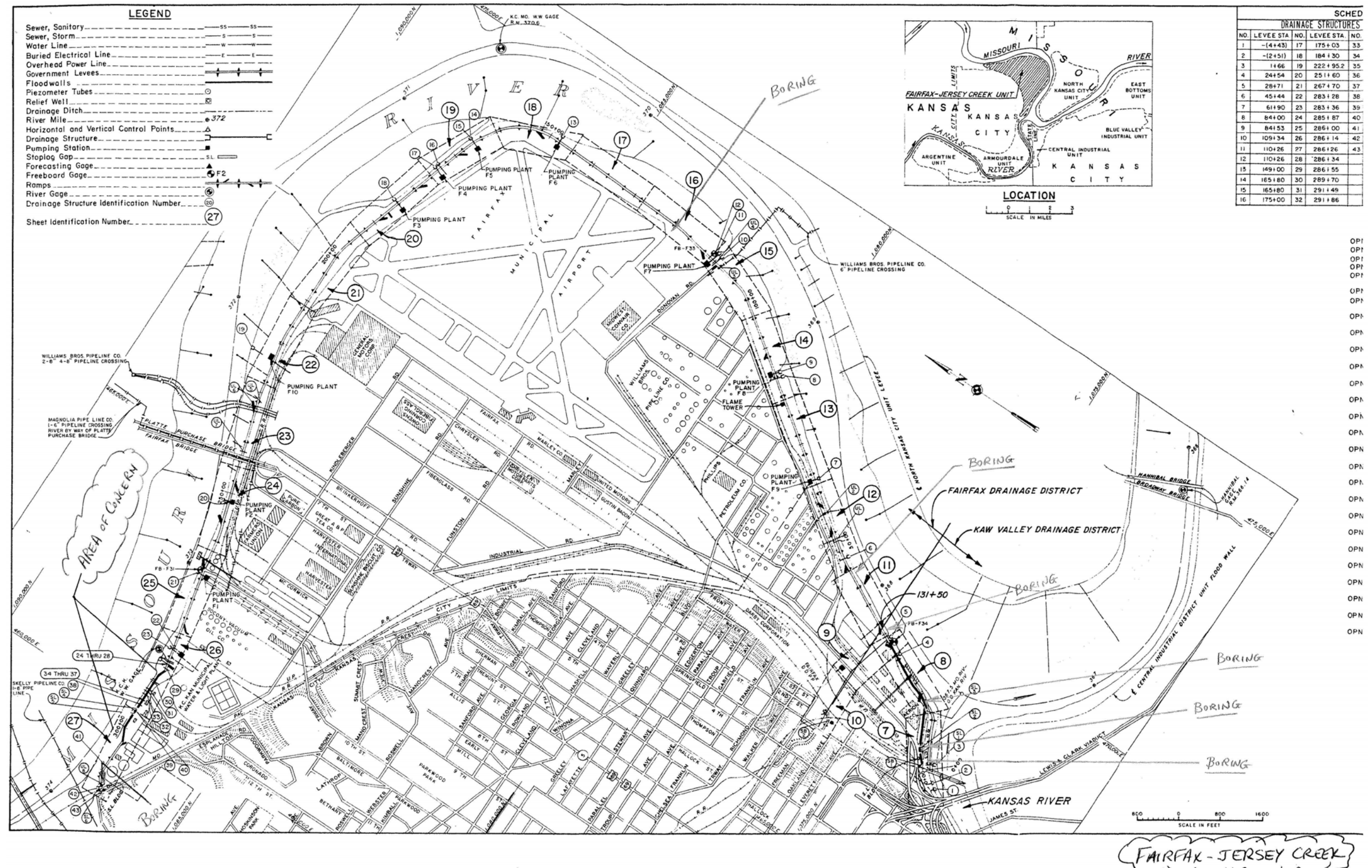
## **A-7.5 REFERENCES**

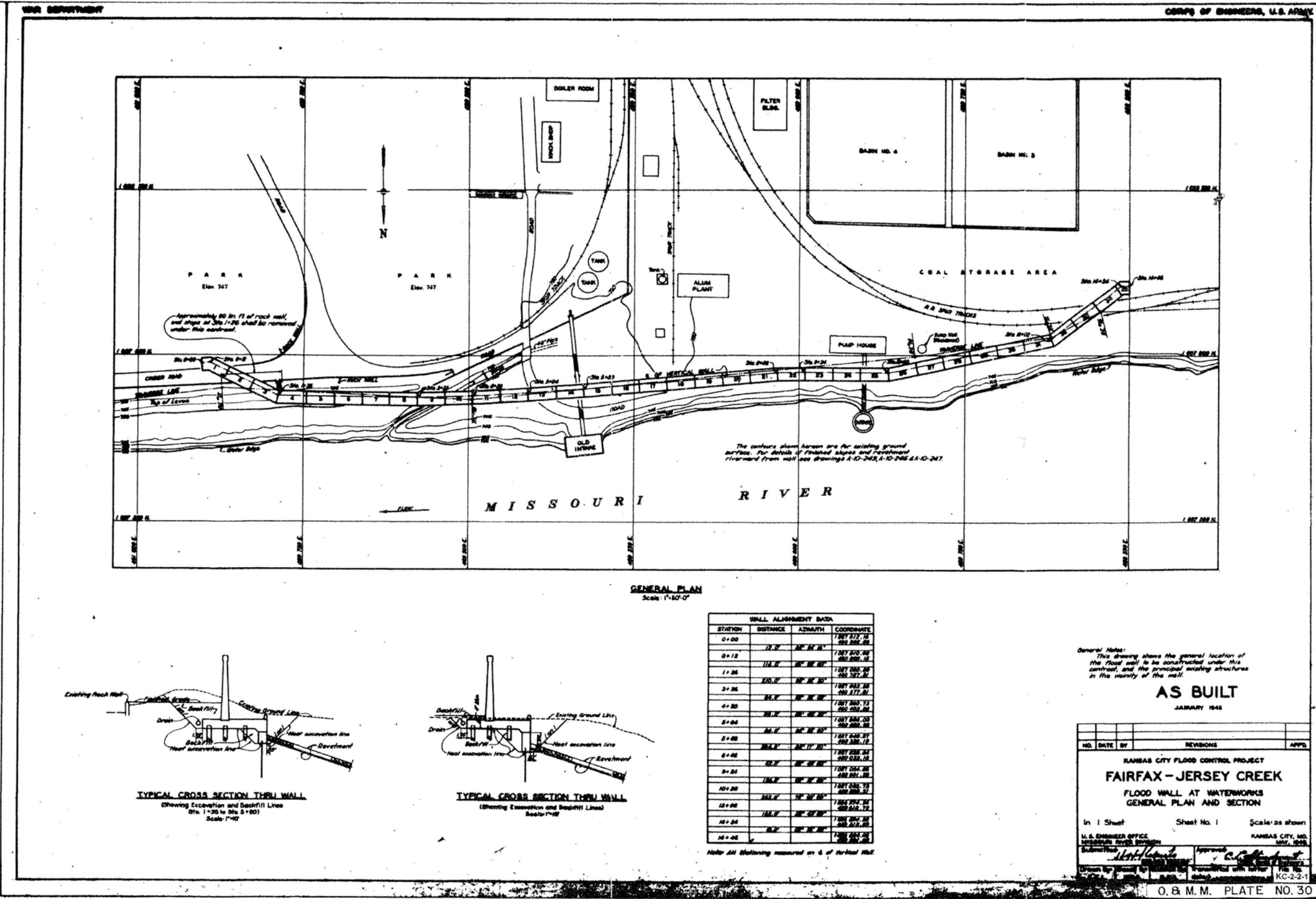
1. Fairfax Jersey Creek Levee Unit, Definite Project Report, December 1938.
2. Fairfax Jersey Creek Levee Unit, Definite Project Report, Raising the Existing Flood Protection, September 1945.
3. Fairfax Jersey Creek Levee Unit, Analysis of Design Jersey Creek Sewer, March 1951.
4. Fairfax Jersey Creek Levee Unit, Supplement on Interior Drainage, September 1952.
5. Fairfax Jersey Creek Levee Unit, Analysis of Design Modifications to Existing Flood Protection, September 1952.
6. Fairfax Jersey Creek Levee Unit, Analysis of Design Pump Plant F2, F3, F6 and F9, December 1952.
7. Fairfax Jersey Creek Levee Unit, Analysis of Design Pump Plant F4, F5, F7 and F8, June 1953.
8. Fairfax Jersey Creek Levee Unit, Design Memorandum No. 1, August 1953.
9. Fairfax Jersey Creek Levee Unit, Design Memorandum No. 2 - Pump Plant F1, October 1953.
10. Fairfax Jersey Creek Levee Unit, Analysis of Design Station 1+54 to Station 29+77 Floodwall and I-wall, February 1954.
11. Fairfax Jersey Creek Levee Unit, Record Drawings – Operations and Maintenance Manual Volume One, May 1941 to October 1952.
12. Fairfax Jersey Creek Levee Unit, Record Drawings – Operations and Maintenance Manual Volume Two, November 1953 to July 1961.
13. Fairfax Jersey Creek Levee Unit, Record Drawings – Levee and Appurtenances February 1954.
14. Fairfax Jersey Creek Levee Unit, Record Drawings – Levee and Appurtenances, Appendix I, May 1959.
15. Fairfax Jersey Creek Levee Unit, Record Drawings – Pump Plants, Appendix II, September 1956.

16. Fairfax Drainage District – Flood Wall Pile Investigation Report, October 2003.
17. “Shear Strength Correlations for Geotechnical Engineering”. Duncan, Horz, and Yang, August 1989.
18. APILE Plus 3.0 for Windows, Reese, Wang, and Arrellaga, 1998.
19. “Design of Pile Foundations”, U.S. Army Corps of Engineers, Engineering Manual EM 1110-2-2906, 15 January 1991.
20. “Risk Analysis in Geotechnical Engineering for Support of Planning Studies”, U.S. Army Corps of Engineering, Engineer Technical Letter, ETL 1110-2-556, 28 May 1999.
21. “Sheet Pile Wall and Levee Evaluation – Jersey Creek Outfall to Wharf Structure, Kansas City, Kansas”, URS Corporation, October 7, 2004.

## **A-7.6 SUPPLEMENTAL EXHIBITS**

**EXHIBIT A-7.1**

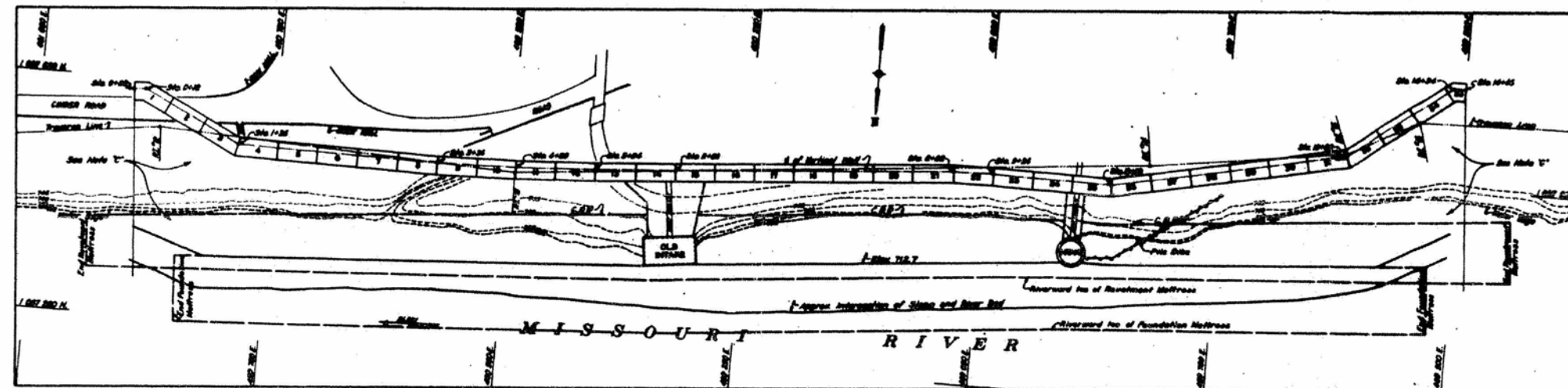




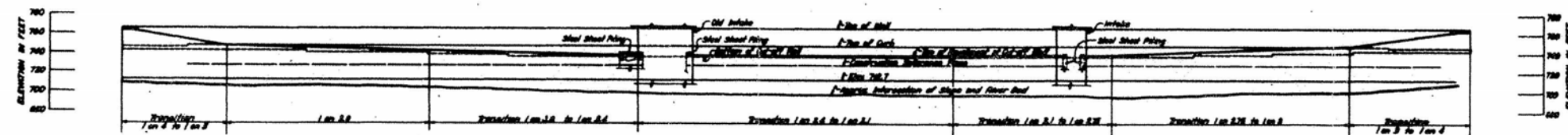
**EXHIBIT A-7.3**

WAR DEPARTMENT

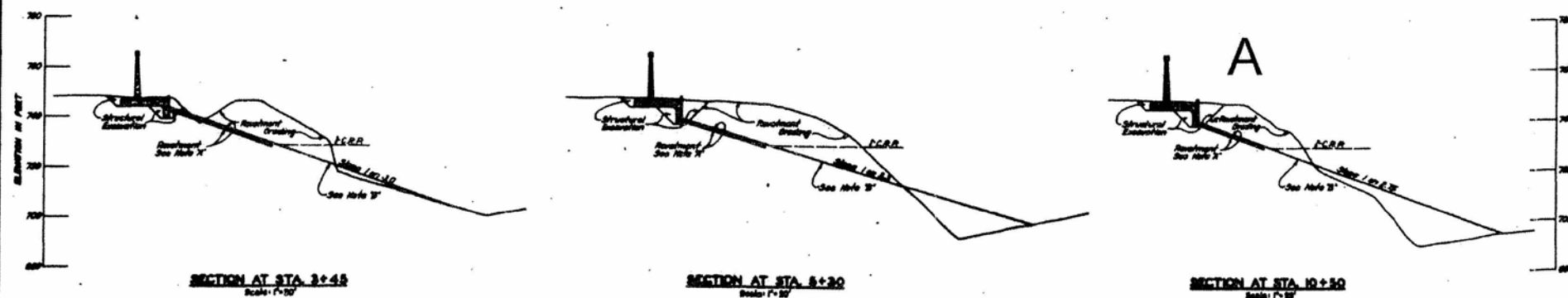
CORPS OF ENGINEERS, U. S. ARMY



**PLAN**  
Scale: 1" = 50'



**ELEVATION**  
Scale: 1" = 30'



Note 2)  
Benchmark varies from 14" thick of C.R. to 10"  
of cut-off wall. Stone to be laid on 4" blanket of  
crushed rock or gravel.  
Note 3)  
For details of Wall and Lumber Mattress see  
Drawings A-12-346 and A-12-347.  
Note 4)  
Grading for connection to levee which is being  
constructed under another contract to be as  
directed by Contracting Officer.

**AS BUILT**

JANUARY 1948

1	1-10-68	URG	Mr. J. J. [unclear] - Mr. J. J. [unclear]	APPRO
NR	6-27-68	EC	REVISIONS	APPRO

KANSAS CITY FLOOD CONTROL PROJECT

# FAIRFAX - JERSEY CREEK

BANK PROTECTION AT WATERWORKS  
PLAN, ELEVATION AND SECTIONS

In 1 Sheet Sheet No. 1 Scale: as shown

U.S. ENGINEERING OFFICE  
KANSAS CITY DISTRICT

KANSAS CITY MISSOURI

Checked by: [Signature]	Reviewed by: [Signature]	Approved: [Signature]
Drawn by: [Signature]	Designed by: [Signature]	File No. A-10-243

O. & M. M. PLATE NO. 75

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

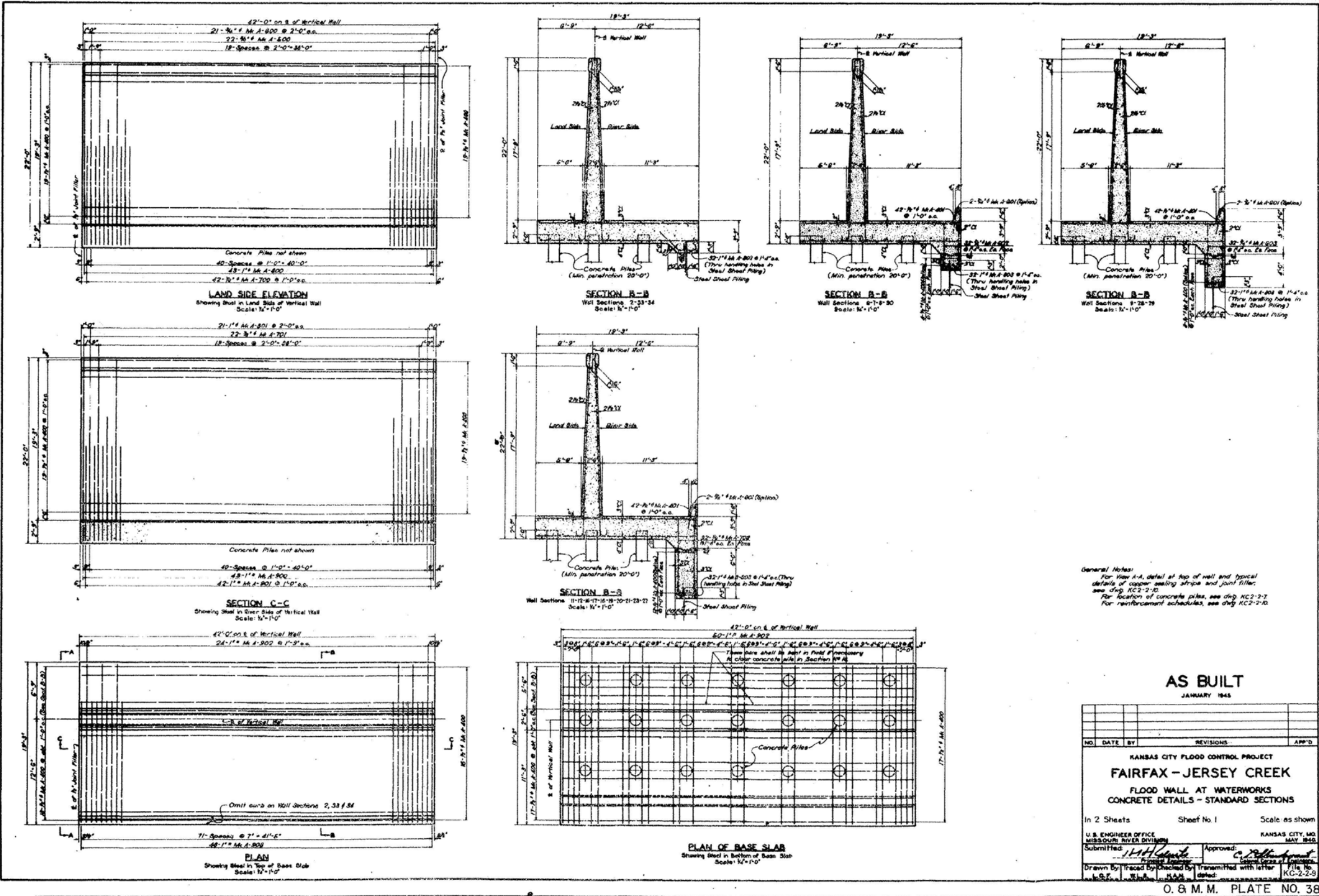
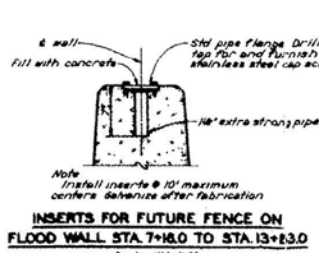
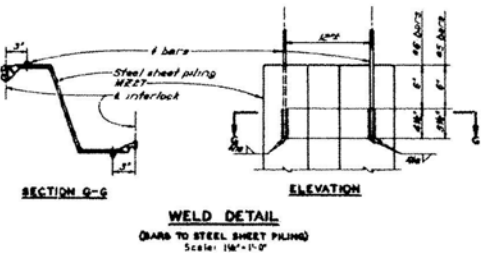
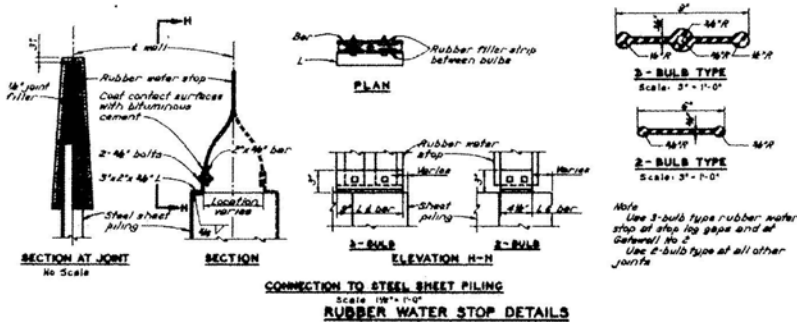
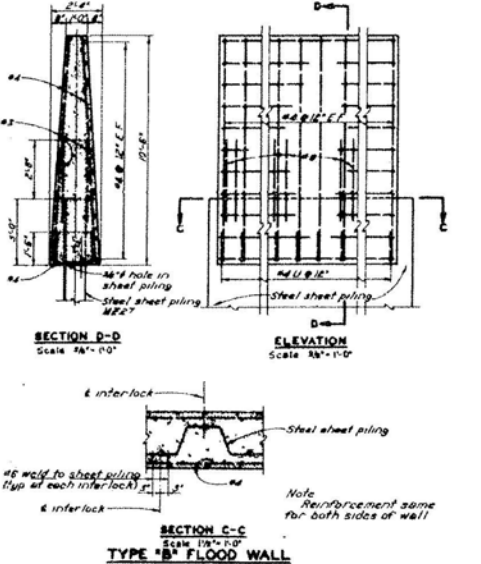
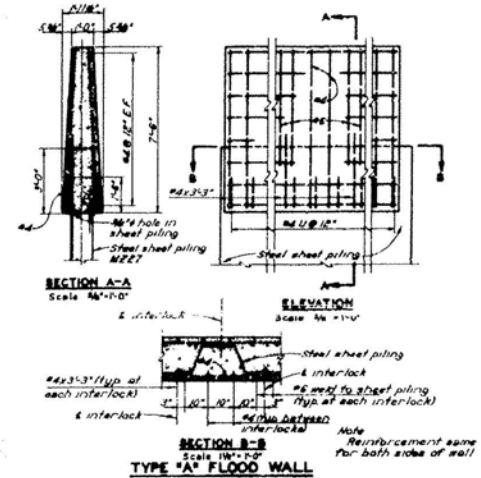
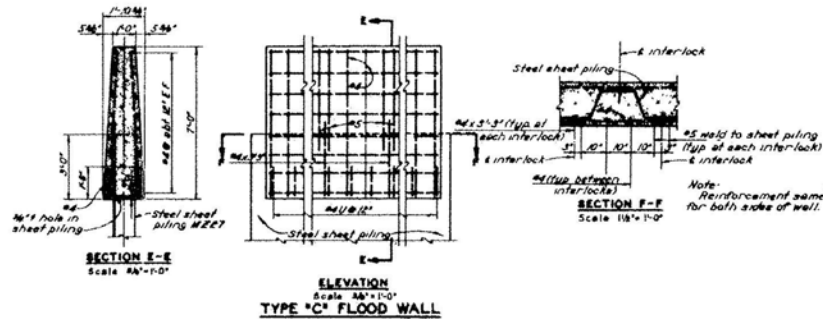
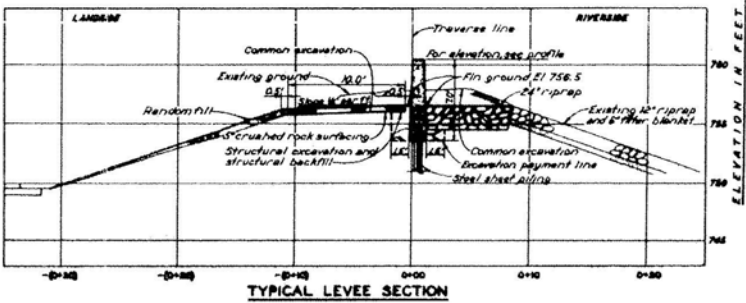
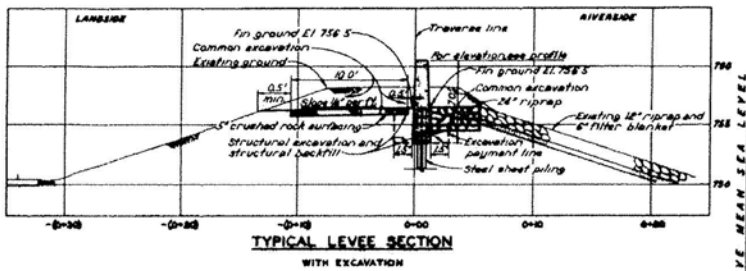
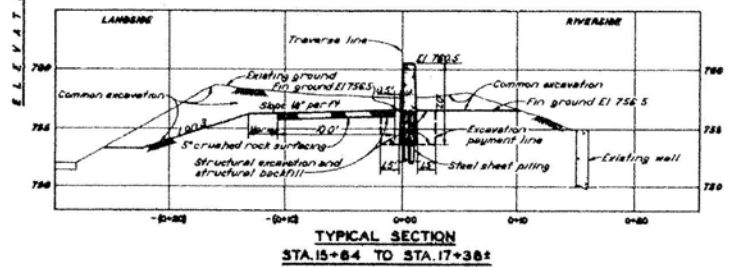
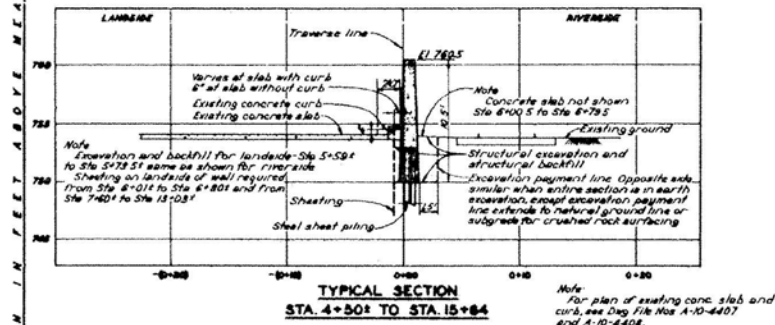
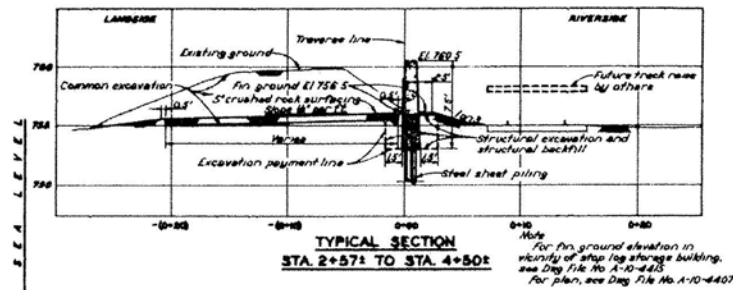


EXHIBIT A-7.5

CORPS OF ENGINEERS

U.S. ARMY



NOTE:  
All structural backfill landward of flood wall shall be pervious material.  
All structural backfill riverward of flood wall shall be random material.  
All exposed edges of concrete shall be chamfered.  
Clear distance of reinforcement from surface of concrete shall be 2" unless otherwise noted.  
For detail of 6" pipe thru steel sheet piling, see Day file No. A-10-4408.

NO.	DATE	BY	REVISIONS	APPD.
2	7-15-55	E.B.C.	Minor correction for "as built" conditions	R.L.G.
1	4-30-54	E.B.C.	Minor revisions	

KANSAS CITY FLOOD CONTROL PROJECT  
FAIRFAX-JERSEY CREEK UNIT  
MODIFICATIONS TO EXISTING PROTECTION  
TYPICAL LEVEE SECTIONS AND  
FLOOD WALL DETAILS

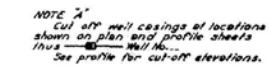
RECORD DRAWING

JULY 1955  
CONTRACT NO. DA-33-038-B-ENG. 2148



In 18 sheets  
OFFICE, DISTRICT ENGINEER  
KANSAS CITY DISTRICT  
Submitted by  
Checked by  
J.M.W. G.P. E.B.C.  
Sheet No. 11  
Scale as shown  
KANSAS CITY, MO.  
FEBRUARY 1954  
Approved  
Chief Engineering Division  
Dist. Engr. Division  
File No.  
A-10-4411

O. & M. M. PLATE NO. 390



1	June '54	JMM	Revised for "As Built" conditions	6/57
NO.	DATE	BY	REVISIONS	APP'D

KANSAS CITY FLOOD CONTROL PROJECT  
FAIRFAX-JERSEY CREEK UNIT  
RAISING EXISTING PROTECTION  
TYPICAL LEVEE SECTIONS

In 32 sheets  
OFFICE, DISTRICT ENGINEER  
KANSAS CITY DISTRICT

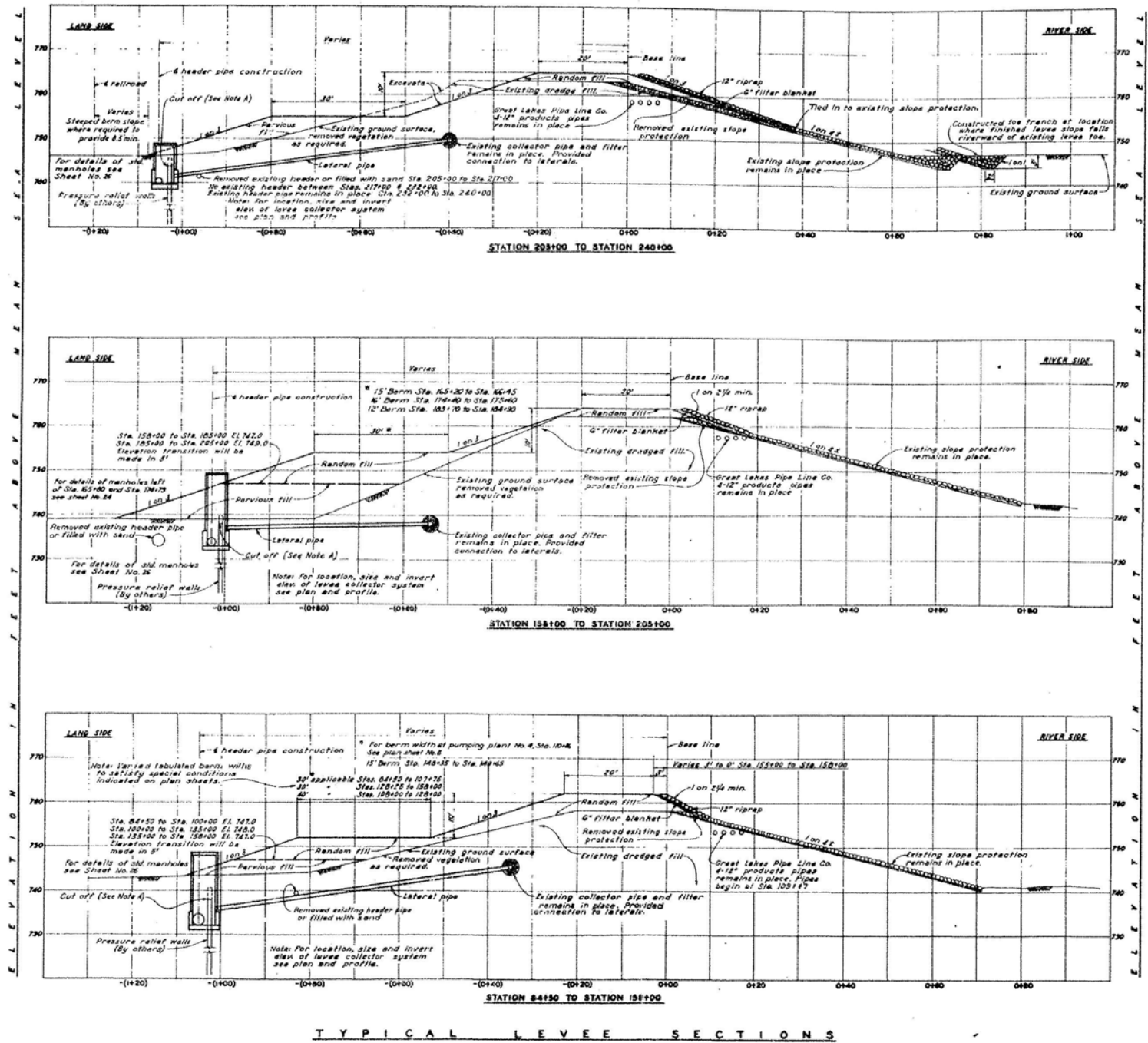
Sheet No. 21

Scale: as shown  
KANSAS CITY, MO.  
MAY 1951

JUNE 1954  
CONTRACT NO. DA-23-028-ENG-868

Submitted: C. W. Brown Recommended: Louis L. Neil Approved: L. J. Binkley  
 Chief, Design Branch Chief, Engineering Division Col. & E. District Engineer  
 Drawn by: Traced by: Checked by: To accompany specifications File No.  
 J.M.B. J.F.N. H.E.B. Dated: MAY 1954 A-10-2591

O. & M. M. PLATE NO. 161



Note: Cut off wall casings at locations shown on plan and profile sheets. See profile for cut-off elevations.

1 June 54 JMM Revised for "As Built" conditions				AKS
NO.	DATE	BY	REVISIONS	APP'D.
KANSAS CITY FLOOD CONTROL PROJECT				
FAIRFAX-JERSEY CREEK UNIT				
RAISING EXISTING PROTECTION				
TYPICAL LEVEE SECTIONS				

RECORD DRAWING

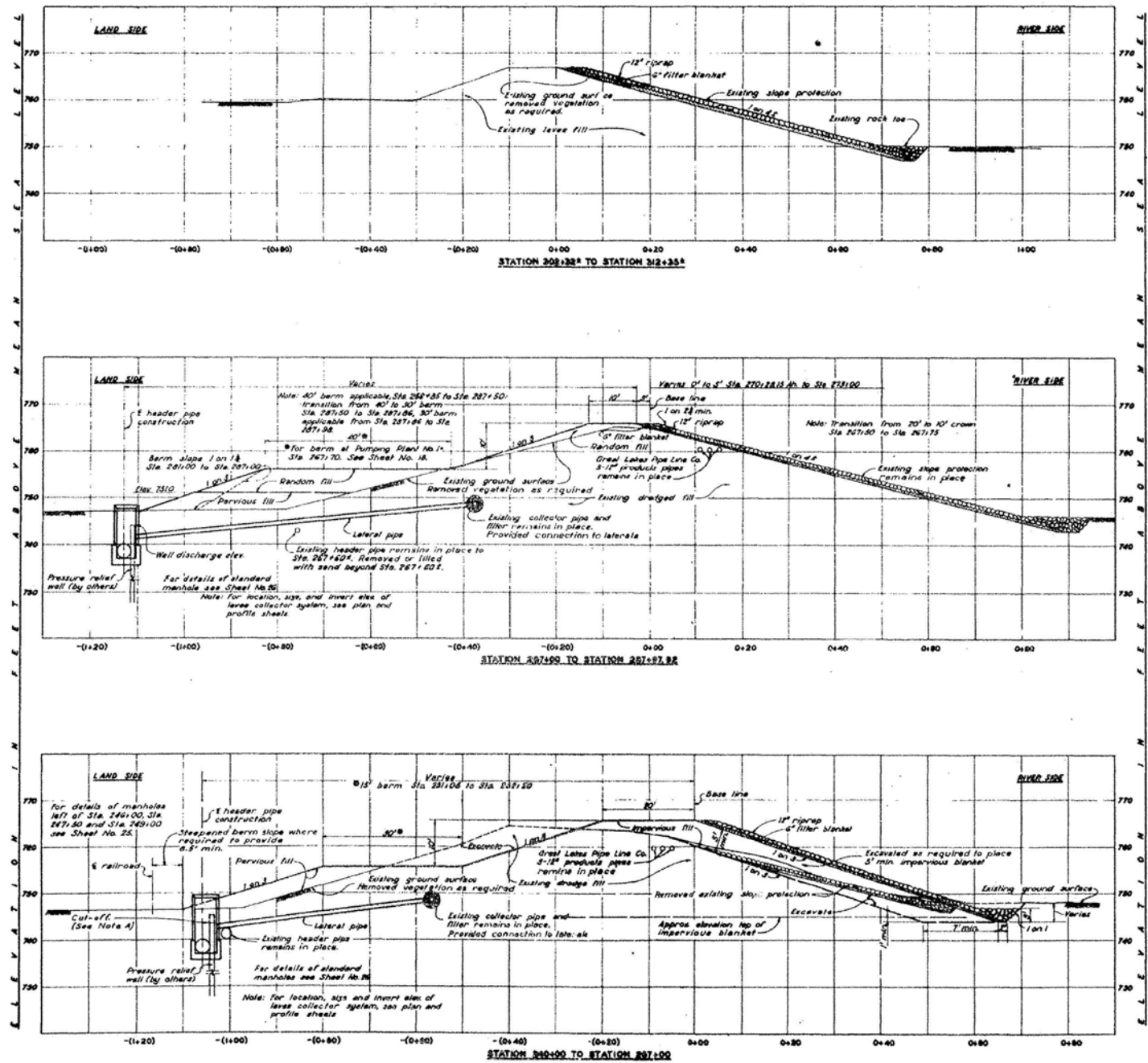
CONTRACT NO. DA-23-028-ENG-000

In 32 sheets  
OFFICE DISTRICT ENGINEER  
KANSAS CITY DISTRICT  
JUNE 1954  
Drawn by: J.W.B.  
Checked by: J.W.B.  
H.E.B.  
Scale: as shown  
KANSAS CITY, MO.  
MAY 1951  
Approved: [Signature]  
Checked by: [Signature]  
File No. A-10-2592

O. & M. M. PLATE NO. 162

CORPS OF ENGINEERS

U.S. ARMY



Note A: Cut off well casings at locations shown on plan and profile sheets. See profile for cut-off elevations.

TYPICAL LEVEE SECTIONS

RECORD DRAWING

JUNE 1964  
CONTRACT NO. DA-23-028-ENG-666

NO.		DATE	BY	REVISIONS	APPRO.
KANSAS CITY FLOOD CONTROL PROJECT FAIRFAX - JERSEY CREEK UNIT RAISING EXISTING PROTECTION TYPICAL LEVEE SECTIONS					
In 32 sheets OFFICE, DISTRICT ENGINEER KANSAS CITY DISTRICT JUNE 1964 CONTRACT NO. DA-23-028-ENG-666					
Sheet No. 23 Scale: as shown KANSAS CITY, MO. MAY 1961					
Submitted: Recommended: Approved: Chief Engineer Division Checked by: To accompany specifications J. M. M. H. E. S. H. E. S. Dated: MAY 1961 A-10-2593					

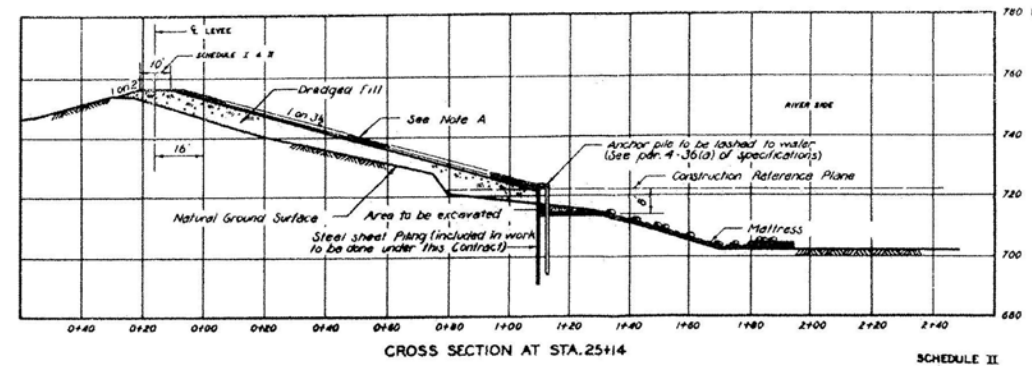
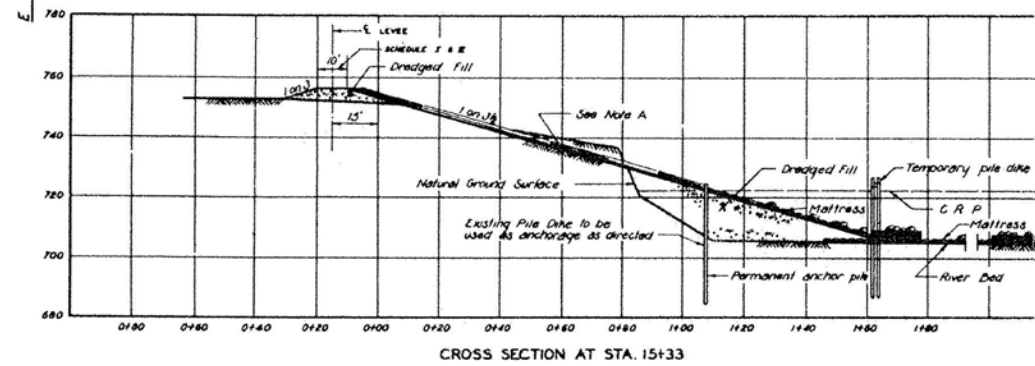
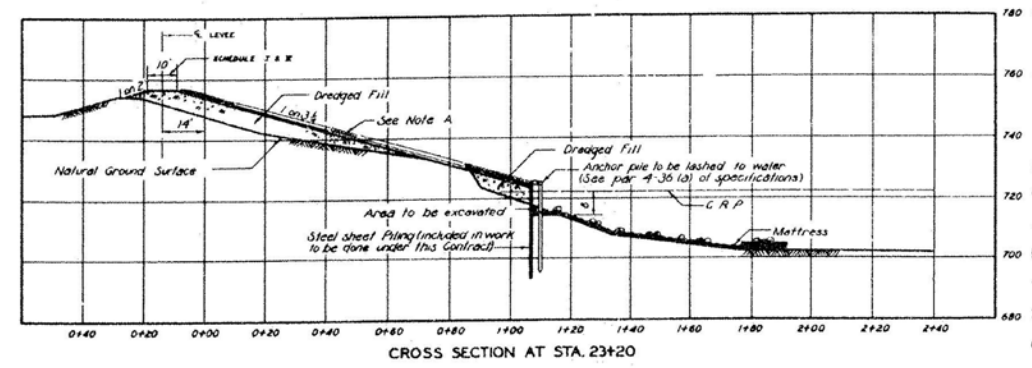
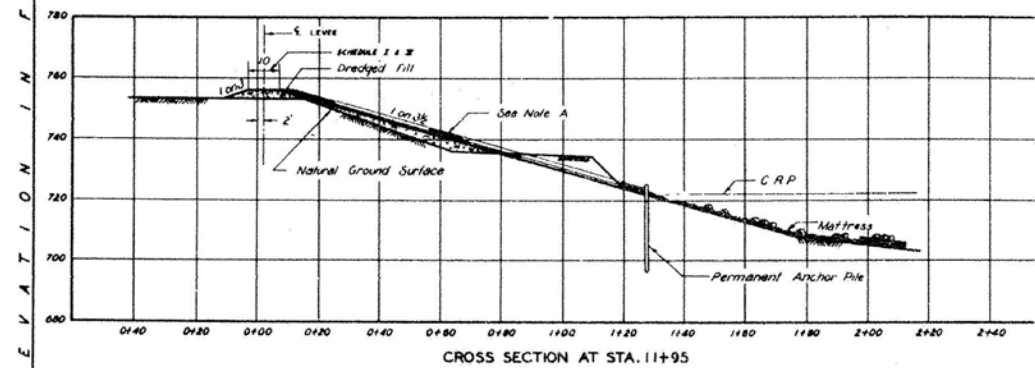
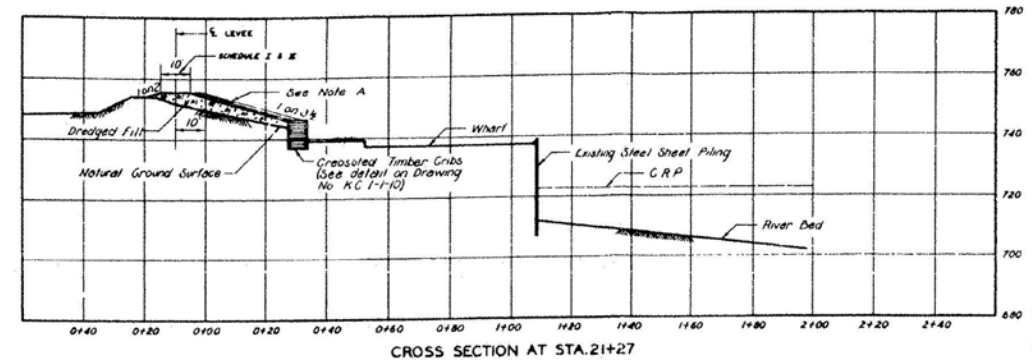
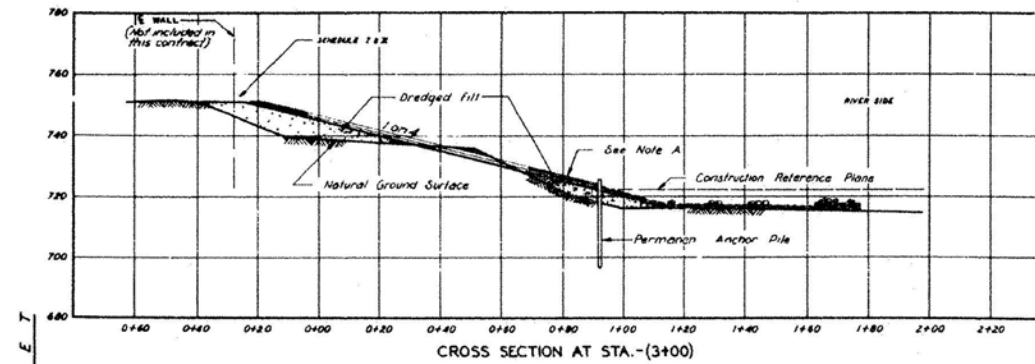
O. & M. M. PLATE NO. 163

**EXHIBIT A-7.9**

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

12



## NOTES

- (A) Revealment varies from 14" thick at C.R.P. to 10" at crown. Stone to be laid on 4" blanket of crushed rock or gravel.

Elevations, referred to mean sea level, are based on U.S. Coast and Geodetic Survey, 1929 General Adjustment.

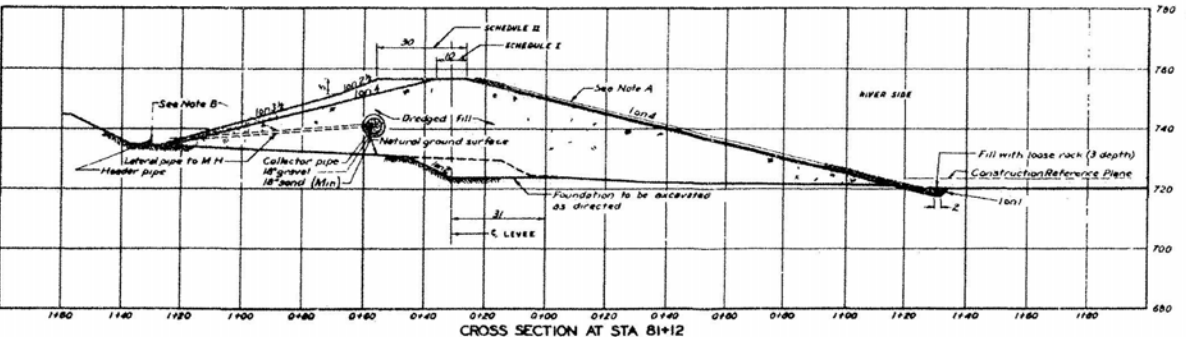
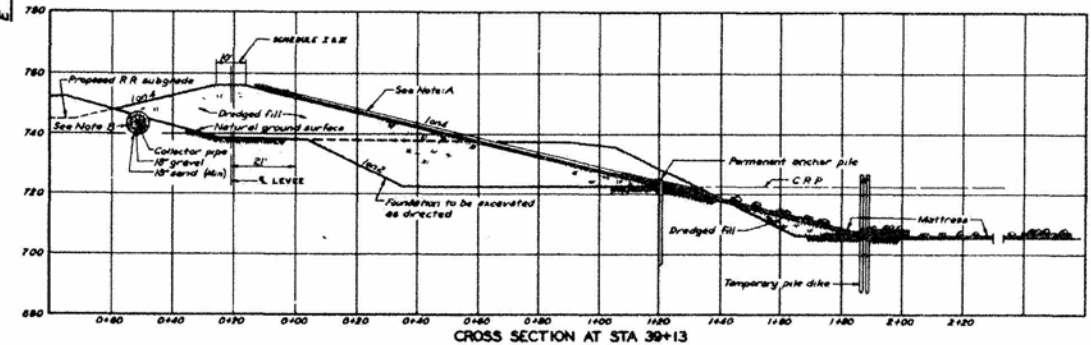
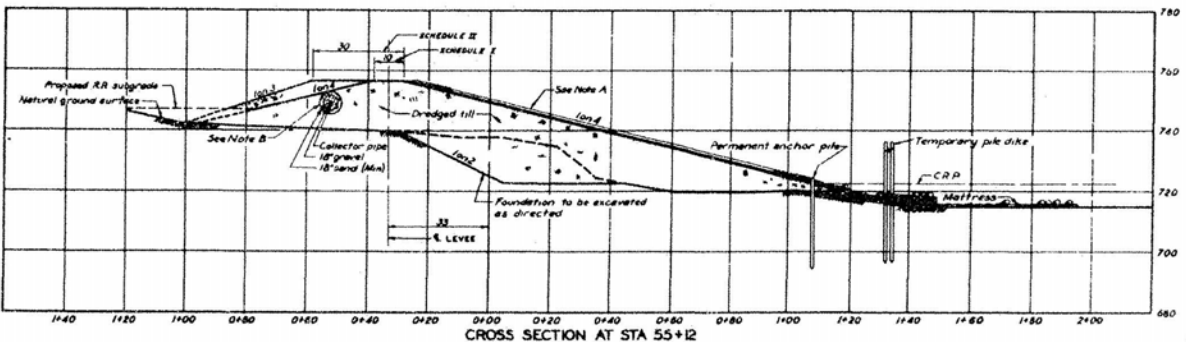
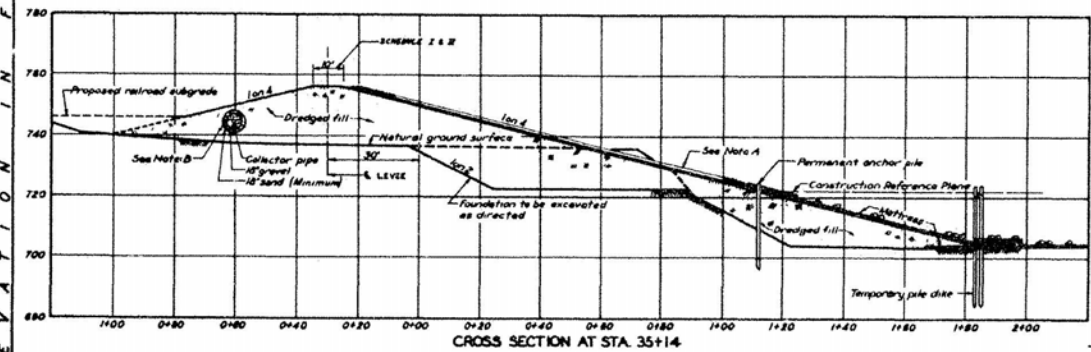
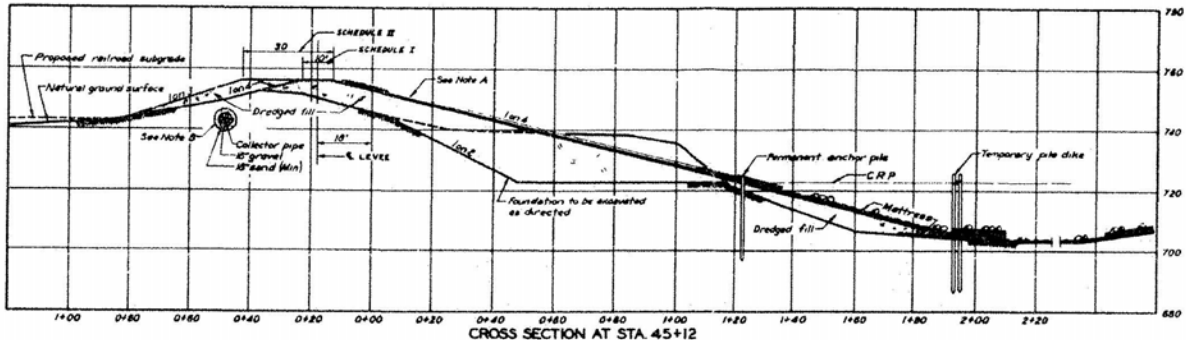
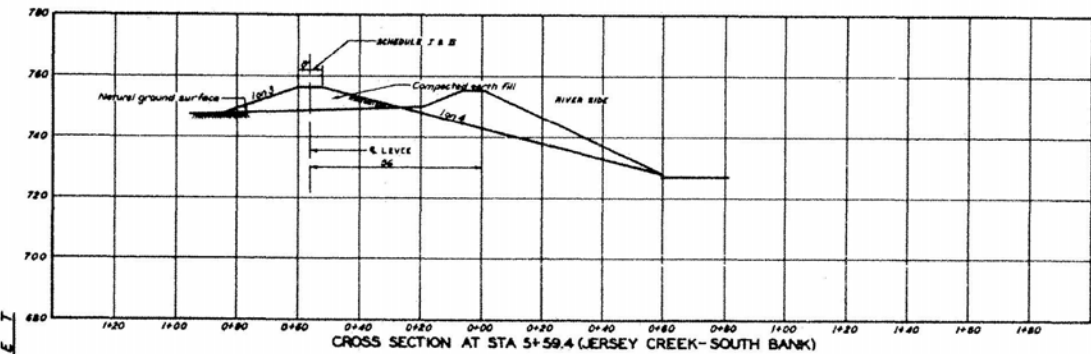
SCHEDULE II  
AS BUILT  
JANUARY 1945

1	1-10-65	JPC	Revised for As Built conditions	<i>[Signature]</i>
NO	DATE	BY	REVISIONS	APPROVED

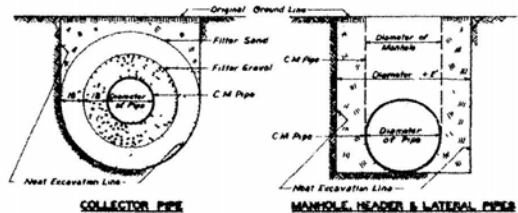
KANSAS CITY FLOOD CONTROL PROJECT  
FAIRFAX - JERSEY CREEK  
FAIRFAX LEVEE  
CROSS SECTIONS

In 3 Sheets Sheet No 1 Scale: as shown  
U.S. ENGINEER OFFICE KANSAS CITY, MO.  
KANSAS CITY DISTRICT  
Submitted: Recommended: Approved:  
Checked by: Transmitted with letter file No:  
Compiled by: Traced by: Checked by: Transmitted with letter file No:  
C.B. R.B. M.L.L. A-127

O. & M. M. PLATE NO. 2



NOTE:  
The dimensions indicated for filter materials are the minimum allowable thicknesses. For convenience of placement, greater thicknesses will be permitted but payment will not be increased as a result of increased thicknesses.



TYPICAL NEAT EXCAVATION LINES FOR PLACEMENT OF  
LEVEE DRAINAGE PIPE BELOW ORIGINAL GROUND LINE  
Not to Scale

NOTES

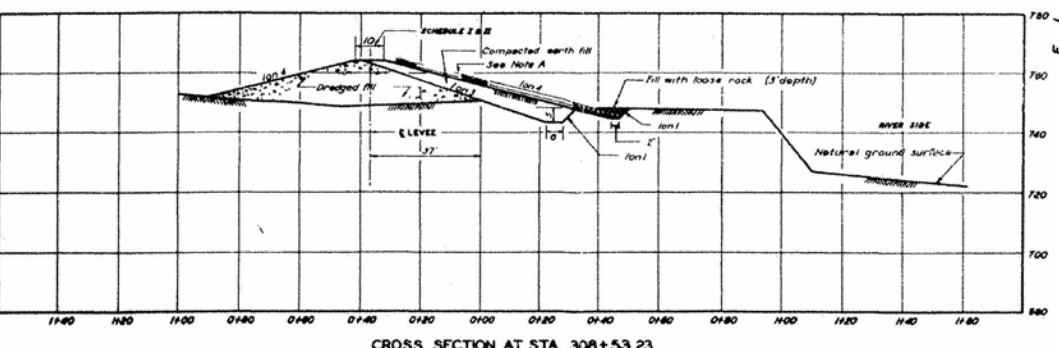
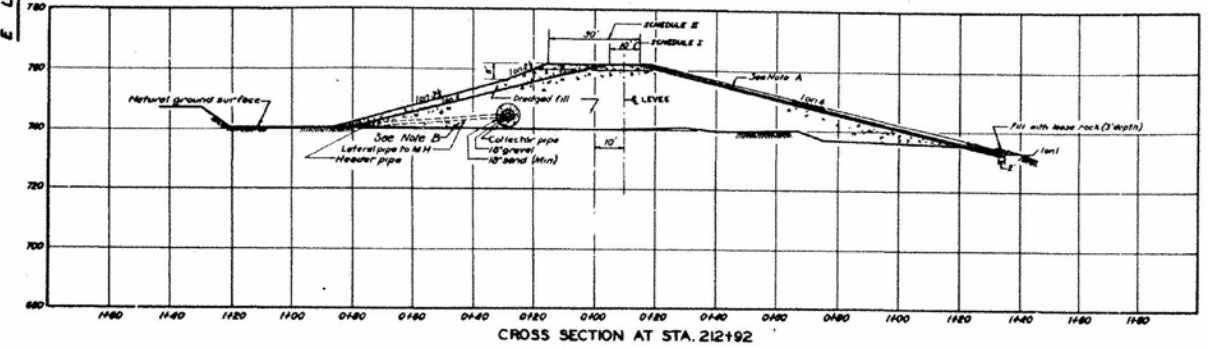
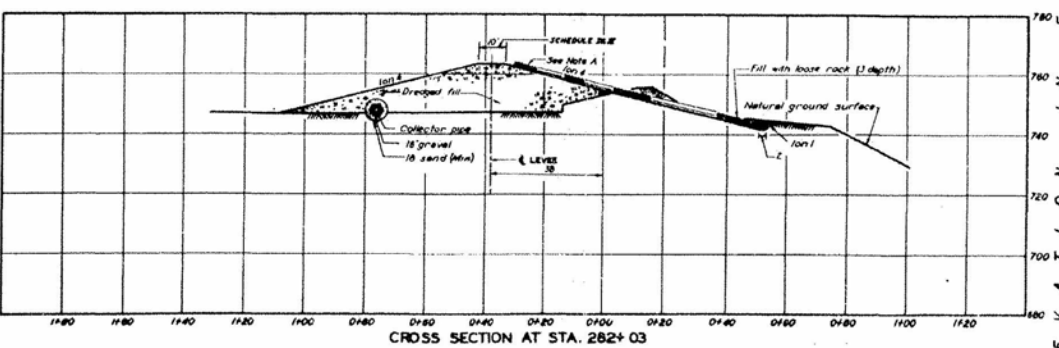
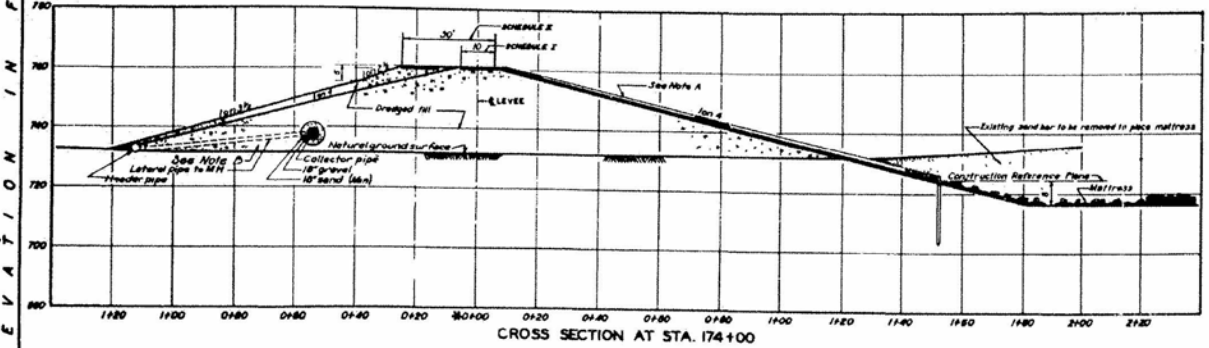
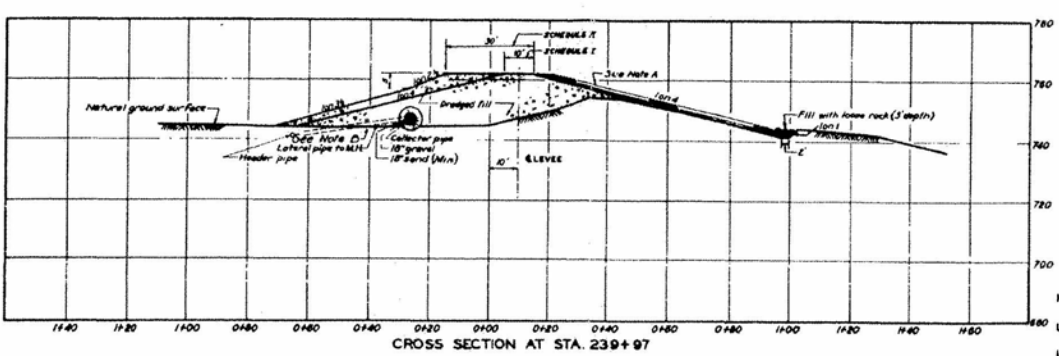
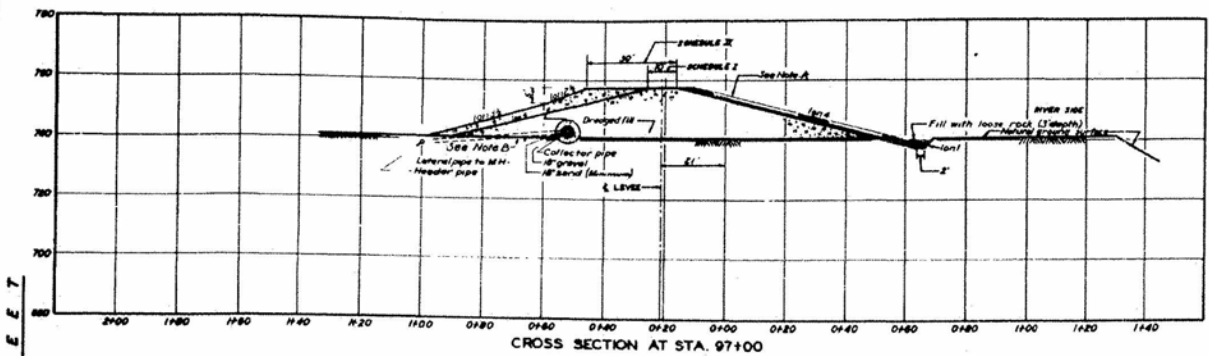
- (A) Revetment varies from 14" thick at C.R.P. to 10" at crown. Stone to be laid on 4" bed of crushed rock or gravel.
  - (B) Location of drainage system is based on Schedule II. For location of manholes, see Plan-profile.
- Elevations, referred to mean sea level, are based on U.S. Coast and Geodetic Survey, 1929 General Adjustment.

SCHEDULE II  
AS BUILT  
JANUARY 1945

KANSAS CITY FLOOD CONTROL PROJECT  
FAIRFAX - JERSEY CREEK  
FAIRFAX LEVEE  
CROSS SECTIONS

In 3 Sheets  
U.S. ENGINEER OFFICE  
KANSAS CITY DISTRICT  
Submitted  
Recommended  
Approved  
Scale as shown  
KANSAS CITY, MO.  
DECEMBER 1939  
A-10-213

NO.	DATE	BY	REVISIONS	APP'D.
1	1-10-45	V.P.C.	Revised for as built conditions	



\* Note:  
Transverse stationing referred  
to 1/2 of 10' crown levee

NOTES

- (A) Revetment varies from 14" thick at C.R.P. to 10" at crown. Stone to be laid on 4" blanket of crushed rock or gravel.
- (B) Location of drainage system is based on Schedule II. For location of manholes, see Plan-profile.

Elevations, referred to mean sea level, are based on U.S. Coast and Geodetic Survey, 1929 General Adjustment.

SCHEDULE II  
AS BUILT  
JANUARY 1945

KANSAS CITY FLOOD CONTROL PROJECT  
FAIRFAX - JERSEY CREEK  
FAIRFAX LEVEE  
CROSS SECTIONS

In 3 Sheets  
U.S. ENGINEER OFFICE  
KANSAS CITY DISTRICT  
Submitted: *[Signature]*  
Recommended: *[Signature]*  
Approved: *[Signature]*

Sheet No. 3  
Scale as shown  
KANSAS CITY, MO.  
DECEMBER, 1939

NO.	DATE	BY	REVISIONS	APPROVED
1	1-10-45	J.C.C.	Revised for as built conditions	<i>[Signature]</i>

## **Site Characterization Maps and Boring Information**

EXHIBIT A-7.12

1

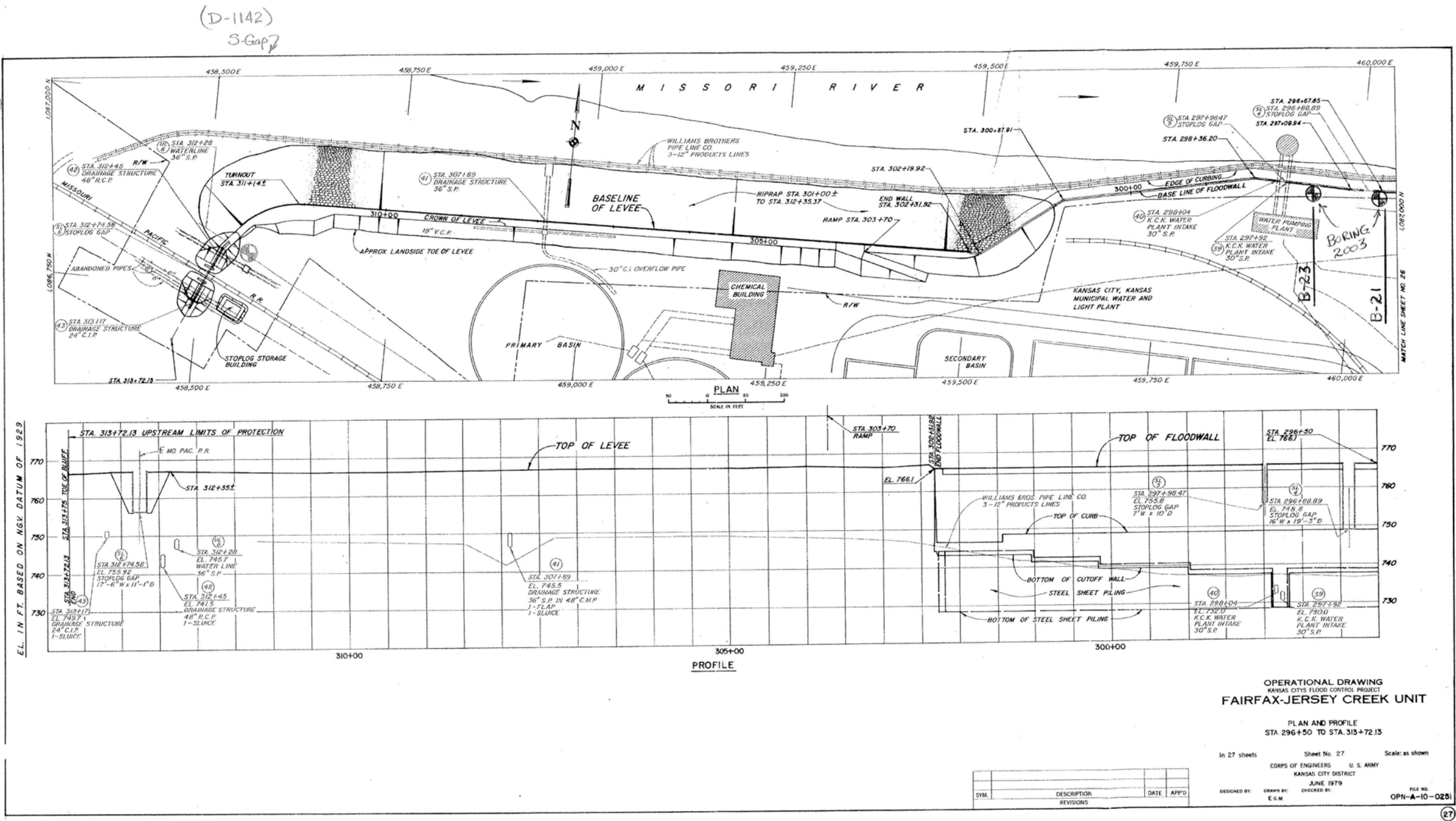
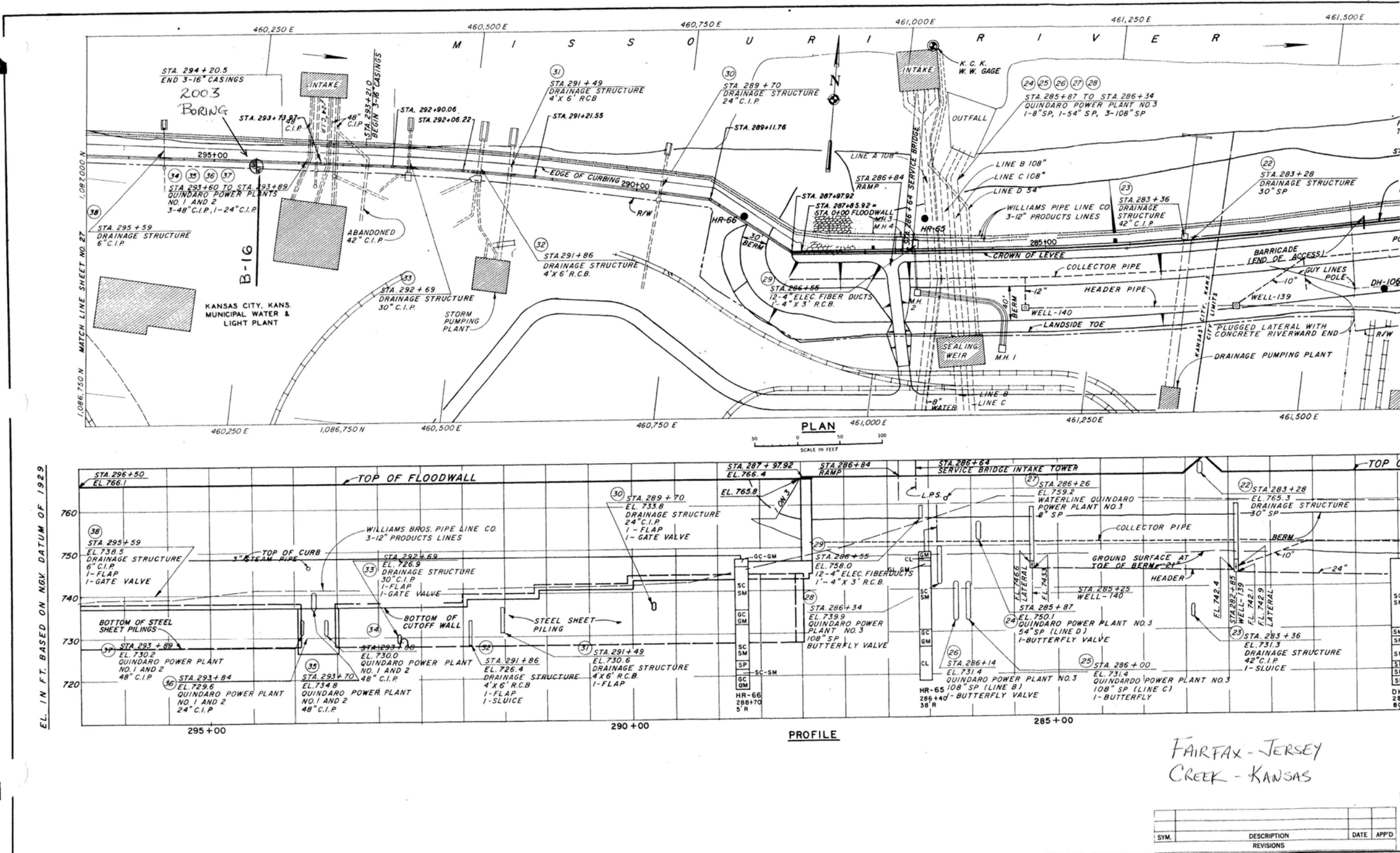


EXHIBIT A-7.13

1



**EXHIBIT A-7.14**  
**Flood Wall Pile Investigation Report**

**Fairfax Drainage District  
Flood Wall Pile Investigation Report  
October 20, 2003**

**U.S. Army Corps of Engineers, Kansas City District  
Contract No. DACW 41-02-D-0006**

**By  
Black & Veatch Special Projects  
Overland Park, Kansas**

October 20, 2003

Page 2

Corps of Engineers, Kansas City District  
Mr. Lamar McKissack

B&V Project 41177  
October 20, 2003

***Attachment A  
Fairfax Flood Wall  
Pile Investigation Report***

**SUBSURFACE EXPLORATION AND  
GEOPHYSICAL SURVEYS  
FAIRFAX FLOOD CONTROL STRUCTURE  
KANSAS CITY, KANSAS**

*Prepared for:*

**U.S. ARMY CORPS OF ENGINEERS,  
KANSAS CITY DISTRICT  
C/O BLACK & VEATCH  
Overland Park, Kansas**

*Prepared by:*

**GEOTECHNOLOGY, INC.  
Overland Park, Kansas**

Geotechnology Job No. 0713201.3211

October 15, 2003

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**SUBSURFACE EXPLORATION AND GEOPHYSICAL SURVEYS**  
**FAIRFAX FLOOD CONTROL STRUCTURE**  
**KANSAS CITY, KANSAS**

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SUBSURFACE EXPLORATION AND GEOPHYSICAL SURVEYS  
FAIRFAX FLOOD CONTROL STRUCTURE  
KANSAS CITY, KANSAS

SECTION I - EXECUTIVE SUMMARY

The executive summary is provided solely for the purposes of overview. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report. Any party who relies on this report must read the full report.

- The project includes conducting an investigation to determine the length of existing piles and the subsurface soil conditions at the Fairfax flood control wall at Monoliths 16, 21 and 23.
- Below the surficial topsoil and silty clay fill, the soil stratigraphy general consists of silty sand underlain by fine to coarse sand.
- After subsurface exploration, the borings were enlarged to install 4-inch diameter PVC pipes for geophysical surveys. Three geophysical methods including parallel seismic, ground penetrating radar (GPR) and magnetic gradient were utilized. The distance from top of pile caps to the bottom of piles were between 21 to 25 feet and the reinforcing steel is between 10 to 13 feet below the top of the pile caps.
- The bottom of the pile cap and the upper 12 to 24 inches of a pile at Monolith 21 were exposed at the face of the excavation to determine the pile shape and cross-sectional dimensions. The exposed pile shows the existing piles are approximately 14-inch diameter, cast-in-place concrete piles (based on a measured pile circumference of 48 inches).

SECTION II - PROJECT DATA

AUTHORIZATION

The services documented in this report were provided in accordance with the terms, conditions and scope of services described in Geotechnology's August 7, 2003 proposal.

PURPOSE AND SCOPE OF SERVICES

The purpose of our services was to conduct subsurface exploration and geophysical surveys for the investigation of the existing piles as defined in our proposal. Briefly, services consisted of site reconnaissance, drilling three borings, laboratory testing, geophysical surveys and data interpretation, and preparation of this report. Important information prepared by The

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Association of Engineering Firms Practicing in the Geosciences (ASFE) for studies of the type is included in Appendix A for your review.

#### PROJECT AND SITE DESCRIPTION

This project included an assessment of the soil conditions adjacent to the existing piling and a determination of the pile length for the Fairfax Drainage District Floodwall in Kansas City, Kansas. The shape and cross sectional dimensions of the existing piles were examined by exposing the pile cap and the upper 12 to 24 inches of pile at Monolith 21, and by exposing the pile caps only at Monoliths 16 and 23. The pile lengths were determined using a total of three types of Borehole Geophysical Surveys (parallel seismic, ground penetrating radar, and magnetic gradient) at Monoliths 16, 21 and 23. This investigation consisted of exposing the pile caps to permit a hammer strike at the top of the concrete to generate a signal source for performing the parallel seismic surveys and performing subsurface explorations. The subsurface explorations included drilling three 50-foot deep bore holes, installing 4-inch diameter PVC pipes, and associated down-hole geophysical surveys. The bore holes were located in the BPU power plant and were drilled adjacent to the existing pile caps.

#### SECTION III - FIELD EXPLORATION AND LABORATORY TESTING

##### FIELD EXPLORATION

The field subsurface exploration consisted of drilling and sampling three borings, designated as Borings B-16, B-21 and B-23, at the approximate locations at Monoliths 16, 21 and 23. All soil borings were drilled and sampled at the above referenced project site from September 22 through 25, 2003. The borings were located in the field by the client.

All borings were drilled to a depth of 50 feet and were drilled using a CME 55 rotary drill rig. The boreholes were drilled using 4-inch flight augers to a depth of 15 feet with rotary wash drilling methods used from a depth of 15 feet to the boring termination depth of 50 feet below ground surface. Standard Penetration Tests (SPT's) were conducted after encountering natural soil deposits (approximately 10 to 13 feet below the ground surface) and using an automatic hammer. Split-spoon samples were obtained at intervals of 2.5 feet to the bottom of each boring. Detailed logs of borings are presented in Appendix B. A bulk sample was obtained during the excavation of the pile caps to determine compaction requirements for the backfilling operation.

An engineer of Geotechnology provided technical direction during field exploration, observed drilling and sampling, assisted in obtaining samples, and prepared descriptive logs of the material encountered. The boring logs represent conditions observed at the time of exploration but have been edited as a result of the laboratory test data as appropriate.

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#### SET UP FOR GEOPHYSICAL SURVEYS

After finishing soil sampling at each boring, the bore holes were re-drilled and enlarged by using 6-1/4 inches inside diameter (I.D.) hollow stem augers to a depth of 15 feet from the ground surface. Eight inch diameter PVC pipe was installed to an approximate depth of 17 feet below the ground surface for use as temporary casing. After the temporary casing was installed, the bore hole was drilled to the termination depth using rotary wash drilling methods and a 7-7/8 inch diameter tri-cone roller bit. Access pipes for geophysical surveys consisting of 4-inch diameter PVC pipes were installed. The annular space between the bore hole and the 4-inch pipe was grouted. The access pipes were backfilled with grout after conducting three geophysical surveys.

#### LABORATORY TESTING

Laboratory testing was performed on the soil samples to determine index properties of the soil and recommended shear strength parameters for re-analysis of pile capacity. Moisture contents, Atterberg limits tests and sieve analyses were performed on selected samples. Results of these laboratory tests are presented on the boring logs and Appendix C. A proctor test was performed on the bulk sample. Summary results are presented in Appendix C.

### SECTION IV - SUBSURFACE CONDITIONS

#### STRATIGRAPHY

Below the surficial 6 inches of top soil, generally 9.5 to 12.5 feet of silty clay fill overlies 10 to 18 feet of very loose to medium dense brown to gray silty sand with intermediate layers of soft, brown and gray clayey silt and silty clay. Below the silt sand, medium dense to dense gray to dark gray silty sand and fine to coarse poorly-graded and well-graded sand were encountered to boring termination at a depth of 50 feet.

#### GROUNDWATER

Groundwater levels were not observed in the borings due to rotary wash drilling methods being used. No groundwater was encountered while drilling with the flight augers to a depth of 15 feet below ground surface.

#### RECOMMENDED SHEAR STRENGTH PARAMETERS

Soil strength parameters for long-term conditions, including cohesion and angle of internal friction were estimated based on the in-situ SPT values. Buoyant unit weight should be used in any computation of allowable pile capacity based on the assumption that the design



APPENDIX B

DETAILED LOGS OF BORINGS B-16, B-21 AND B-23  
BORING LOG: TERMS AND SYMBOLS

Surface Elevation _____		Completion Date: <u>9/24/03</u>		GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/ROD	SAMPLES	SHEAR STRENGTH, tsf		
Datum _____		$\Delta$ - UU/2 $\circ$ - QU/2 $\square$ - SV 0.5    1.0    1.5    2.0    2.5							
DEPTH IN FEET	DESCRIPTION OF MATERIAL		STANDARD PENETRATION RESISTANCE (ASTM D 1586)						
			▲ N-VALUE (BLOWS PER FOOT)						
		WATER CONTENT, %							
		PLI 10    20    30    40    50    LL							
0	Topsoil - 6 inches								
5	FILL : brown, silty SAND, with brick and concrete fragments								
10									
15	Very loose to loose, gray, silty SAND to sandy SILT - SM to ML		4-2-2	SS1					
20	Soft, gray, silty CLAY, trace fine sand - CL		2-1-2	SS2					
25	Loose to very loose, gray, silty SAND - SM		5-5-3	SS3					
30	layers of brown silty CLAY and clayey SAND		4-4-4	SS4					
35			2-0-2	SS5					
40	Loose, gray, poorly-graded fine to medium, SAND, trace layers of gray silty sand - (SP)		5-3-2	SS6					
45			4-7-7	SS7					
<b>GROUNDWATER DATA</b> <input checked="" type="checkbox"/> FREE WATER NOT ENCOUNTERED DURING DRILLING			<b>DRILLING DATA</b> <u>6.25</u> AUGER <u>      </u> HOLLOW STEM WASHBORING FROM <u>13.5</u> FEET DWB DRILLER <u>YAW</u> LOGGER <u>CME 55HT</u> DRILL RIG HAMMER TYPE <u>Auto</u>						
REMARKS:			Drawn by: <u>yaw</u> Ckd. by: <u>yaw</u> App'd. by: <u>W.C.S.</u> Date: <u>9/25/03</u> Date: <u>10/15/03</u> <u>W.C.S.</u> <div style="text-align: center;">   <b>GEOTECHNOLOGY, INC.</b>  ENGINEERING AND ENVIRONMENTAL SERVICES  ST. LOUIS • COLUMBIAVILLE • KANSAS CITY </div>						
			Fairfax Flood Control Structure						
			LOG OF BORING: B-16						
			Project No. 0713201.3211						

NOTE: STRATIFICATION LINES REPRESENT  
 AND THE TRANSITION MAY BE GRADUAL.  
 LOG OF BORING 2002    132 FAIRFAX GP2 GT INC 053301 GPJ 10/15/03

Surface Elevation \_\_\_\_\_  
 Datum \_\_\_\_\_

Completion Date: 9/24/03

SHEAR STRENGTH, tsf		
Δ - UU/2	○ - QU/2	□ - SV
0.5	1.0	1.5
2.0	2.5	

DRY UNIT WEIGHT (pcf)  
 SPT BLOW COUNTS  
 CORE RECOVERY/RCD

STANDARD PENETRATION RESISTANCE (ASTM D 1586)		
▲ N-VALUE (BLOWS PER FOOT)		
WATER CONTENT, %		
PL		LL
10	20	30
40	50	

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	SAMPLES	TEST RESULTS
	Medium dense to dense, gray, poorly-graded medium to coarse, SAND - SP ( <i>continued</i> )		8-9-8 SS8	
			7-10-22 SS9	
35	Medium dense, gray, poorly-graded fine to medium, SAND - SP		8-9-8 SS10	
	Medium dense, gray, poorly-graded medium to coarse, SAND - SP		7-10-10 SS11	
40			9-9-10 SS12	
	Medium dense to dense, gray, poorly-graded fine, SAND with silt - (SP-SM)		15-22-15 SS13	
45			7-12-11 SS14	
	Medium dense, gray, poorly-graded fine to medium, SAND - SP		7-10-10 SS15	
50	Boring terminated at 50 feet			
55				

**GROUNDWATER DATA**  
☒ FREE WATER NOT ENCOUNTERED DURING DRILLING

**DRILLING DATA**  
6.25 AUGER           HOLLOW STEM  
 WASHBORING FROM 13.5 FEET  
DWB DRILLER   YAW LOGGER  
CME 55HT DRILL RIG  
 HAMMER TYPE Auto

Drawn by: yaw	Ch'd by: yaw	App'd by: yaw
Date: 9/25/03	Date: 10/15/03	10/16/03

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**CONTINUATION OF LOG OF BORING: B-16**

Project No. 0713201.3211

**REMARKS:**

Surface Elevation _____		Completion Date: <u>9/22/03</u>		GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/ROD	SAMPLES	SHEAR STRENGTH, tsf		
Datum _____		$\Delta$ - UU/2 $\circ$ - QU/2 $\square$ - SV 0.5    1.0    1.5    2.0    2.5							
DEPTH IN FEET	DESCRIPTION OF MATERIAL	STANDARD PENETRATION RESISTANCE (ASTM D 1586)							
		$\blacktriangle$ N-VALUE (BLOWS PER FOOT) WATER CONTENT, % PLI 10    20    30    40    50    LL							
	Topsoil - 6 inches FILL : brown, silty SAND with brick and concrete fragments and toe drain								
5									
10	Very loose to medium dense, brown to gray, silty SAND - SM	3-2-2-3	SS1	$\blacktriangle$	$\bullet$				
15		20-7-5	SS2	$\blacktriangle$					
	Stiff, brown to gray, sandy SILT - ML	8-2-3	SS3	$\blacktriangle$	$\bullet$				
20		2-6-4	SS4	$\blacktriangle$					
	Medium dense, brown to gray, silty SAND - SM	9-9-9	SS5	$\blacktriangle$					
25		6-8-7	SS6	$\blacktriangle$	$\bullet$				
	Medium dense, brown to gray, poorly-graded fine, SAND with silt - SP-SM	7-10-10	SS7	$\blacktriangle$					
		7-7-11	SS8	$\blacktriangle$	$\bullet$				
	Medium dense, gray, poorly-graded fine, SAND, trace silt - SP								

<b>GROUNDWATER DATA</b> X FREE WATER NOT ENCOUNTERED DURING DRILLING		<b>DRILLING DATA</b> <u>6.25</u> AUGER <u>    </u> HOLLOW STEM WASHBORING FROM <u>10</u> FEET <u>DWB</u> DRILLER <u>JAW</u> LOGGER <u>CME 55HT</u> DRILL RIG HAMMER TYPE <u>Auto</u>		Drawn by: <u>yaw</u> Date: <u>9/25/03</u>	CK'd. by: <u>yaw</u> Date: <u>10/15/03</u>	App'd. by: <u>ws</u> Date: <u>10/16/03</u>
REMARKS:				 <b>GEOTECHNOLOGY, INC.</b> ENGINEERING AND ENVIRONMENTAL SERVICES ST. LOUIS • COLENSVILLE • KANSAS CITY		
				<b>Fairfax Flood Control Structure</b>		
				<b>LOG OF BORING: B-21</b>		
				Project No. 0713201.3211		

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Surface Elevation \_\_\_\_\_  
 Datum \_\_\_\_\_

Completion Date: 9/23/03

**GRAPHIC LOG**

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RCD	SAMPLES	SHEAR STRENGTH, tsf		
				Δ - UU/2	○ - QU/2	□ - SV
				STANDARD PENETRATION RESISTANCE (ASTM D 1586)		
				▲ N-VALUE (BLOWS PER FOOT)		
				WATER CONTENT, %		
				PLI	10 20 30 40 50 I.L.	
0 - 5	Topsoil - 6 inches FILL : brown, silty sand					
5 - 15	Soft, brown, silty CLAY, trace fine sand - (CL)	1-2-1	SS1	▲		
15 - 18	Soft, brown, clayey SILT - ML	0-0-4	SS2	▲		
18 - 20	Soft, brown, clayey SILT - ML	1-3-4	SS3	▲		
20 - 25	Loose, brown, silty SAND - SM	4-5-5	SS4	▲		
25 - 30	Loose, brown, poorly-graded medium, SAND, trace gray sandy silt - SP	3-2-2	SS5	▲		
30 - 35	Dense to medium dense, gray, poorly-graded fine to medium, SAND - SP	14-16-16	SS6		▲	
35 - 40		7-9-8	SS7	▲		

**GROUNDWATER DATA**  
 X FREE WATER NOT ENCOUNTERED DURING DRILLING


**DRILLING DATA**  
 6.25 AUGER \_\_\_\_\_ HOLLOW STEM  
 WASHBORING FROM 13.5 FEET  
 DWB DRILLER YAW LOGGER  
 CME 55HT DRILL RIG  
 HAMMER TYPE Auto

Drawn by: yaw    Ckd. by: yaw    App'd. by: yaw  
 Date: 9/25/03    Date: 10/15/03    10/16/03  
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**Fairfax Flood Control Structure**  
**LOG OF BORING: B-23**  
 Project No. 0713201.3211

**REMARKS:**

Surface Elevation _____		Completion Date: <u>9/23/03</u>		GRAPHIC LOG DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD SAMPLES		SHEAR STRENGTH, tsf Δ - UU/2    ○ - QU/2    □ - SV 0.5    1.0    1.5    2.0    2.5					
Datum _____		DESCRIPTION OF MATERIAL  Dense to medium dense, gray, poorly-graded fine to medium, SAND - SP (continued)  Loose to medium dense, gray, poorly-graded fine, SAND, trace gray silty sand - SP  Medium dense to dense, gray, poorly-graded medium, SAND with silt - SP-SM trace fine gravel  Medium dense, gray, well-graded medium to coarse sand, trace fine gravel - SW  Boring terminated at 50 feet				STANDARD PENETRATION RESISTANCE (ASTM D 1586) ▲ N-VALUE (BLOWS PER FOOT) WATER CONTENT, % PLI 10 20 30 40 50 LL					
DEPTH IN FEET	35					7-7-5	SS8				
						3-4-5	SS9				
40		3-4-7	SS10								
		6-7-5	SS11								
45		6-16-12	SS12								
		7-11-12	SS13								
50		9-9-8	SS14								
		10-10-8	SS15								
55											

<b>GROUNDWATER DATA</b> X FREE WATER NOT ENCOUNTERED DURING DRILLING		<b>DRILLING DATA</b> 6.25 AUGER    _____ HOLLOW STEM WASHBORING FROM <u>13.5</u> FEET DWB DRILLER <u>YAW</u> LOGGER CME 55HT DRILL RIG HAMMER TYPE <u>Auto</u>		Drawn by: yaw    Ckd. by: yaw    App'd. by: <u>[Signature]</u> Date: 9/25/03    Date: <u>10/15/03</u> <u>WILLIAMS</u>	
REMARKS:		 <b>GEOTECHNOLOGY, INC.</b> ENGINEERING AND ENVIRONMENTAL SERVICES ST. LOUIS • COLUMBIA • KANSAS CITY		<b>Fairfax Flood Control Structure</b>  <b>CONTINUATION OF LOG OF BORING: B-23</b>  Project No. 0713201.3211	

## BORING LOG: TERMS AND SYMBOLS

### GENERAL NOTES

- Information on each boring log is a compilation of subsurface conditions based on soil or rock classifications obtained from the field as well as from laboratory testing of samples. The strata lines on the logs may be approximate or the transition between the strata may be gradual rather than distinct. Water level measurements refer only to those observed at the times and places indicated, and may vary with time, geologic condition or construction activity.
- Relative composition and Unified Soil Classification designations are based on visual estimates and are approximate only. If laboratory tests were performed to classify the soil, the unified designation is shown in parenthesis.
- Value given in Unit Dry Weight/SPT Column is either a unit dry weight in pounds per cubic foot, if adjacent to a ST sample designation, or blows per 6-inch increment if adjacent to a SS sample designation.

### ABBREVIATIONS

UU/2	Shear Strength from Unconsolidated - Undrained Triaxial Test (ASTM D2850)
QU/2	Shear Strength from Unconfined Compression Test (ASTM D2168)
SV	Shear Strength from Field Vane (ASTM D2573)
PL	Plastic Limit (ASTM D4318)
LL	Liquid Limit (ASTM D4318)

### LEGEND

CS	Continuous Sampler
GB	Grab Sample Taken From Auger Cuttings Or Wash Water Return
NX	NX Rock Core with Percent Recovery/R.Q.D. Given in Adjacent Column
100	
42	
PST	Three Inch Diameter Piston Tube Sample
SS	Split Spoon Sample (Standard Penetration Test)
ST	Three Inch Diameter Shelby Tube Sample
*	Sample Not Recovered
SV	Field Vane Test

### SPLIT - BARREL SAMPLER DRIVING RECORD

Blow Per Foot (N-Value)	Description
25	25 blows drove sampler 12 inches after initial 6 inches of seating.
75/10"	75 blows drove sampler 10 inches after initial 6 inches of seating.
60/3"	60 blows drove sampler 3 inches during initial 6 inch seating interval.

NOTES: 1. To avoid damage to sampling tools, driving is limited to 60 blows during any six inch interval.  
 2. N-Value (Blow Count) is the standard penetration resistance based on the total number of blows, using a 140-lb. hammer with 30-inch free fall, required to drive a split spoon the last two of three, 6 inch drive increments. (Example: 47/2, N = 7 + 9 = 16). Values are shown as a summation on grid plot and may be shown as 47/2 in Unit Dry Weight-SPT column.

### RELATIVE COMPOSITION

Trace	0-10%
With/Some	11-35%
Soil modifier such as silty, clayey, sandy, etc.	> 35%

### DENSITY OF GRANULAR SOILS

Descriptive Term:	N-Value
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	> 50

### STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength Tons Per Sq. Ft.	Field Test	Approximate N-Value Range
Very Soft	less than 0.12	Thumb will penetrate soil more than 1"	0-1
Soft	0.13 to 0.25	Thumb will penetrate soil about 1"	2-4
Medium Stiff	0.26 to 0.50	Thumb will penetrate soil about 1/4"	5-8
Stiff	0.51 to 1.00	Thumb hardly indents soil	9-15
Very Stiff	1.01 to 2.00	Thumb will not indent soil, but readily indented with thumbnail	16-30
Hard	greater than 2.00	Thumbnail will not indent soil	> 30

### SOIL GRAIN SIZE U.S. STANDARD SIEVE

U.S. STANDARD SIEVE									
12"	3"	3/4"		4	10	40	200		
BOULDERS	COBBLES	GRAVEL		SAND			SILT		CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE			
300	76.2	19.1	4.76	2.00	0.42	0.074			0.002
SOIL GRAIN SIZE IN MILLIMETERS									

### SOIL GRAIN SIZE IN MILLIMETERS

### SOIL STRUCTURE

**Calcareous** — Having appreciable quantities of carbonate.  
**Fissured** — Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.  
**Slickensided** — Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the ease of breaking along these planes.  
**Layer** — Inclusion greater than 3 inches thick.  
**Seam** — Inclusion 1/8 inch to 3 inches thick extending through the sample.

**Parting** — Inclusion less than 1/8 inch thick.  
**Pocket** — Inclusion of material of different texture that is smaller than the diameter of the sample.  
**Interlayered** — Soil samples composed of alternating layers of different soil types.  
**Intermixed** — Soil samples composed of pockets of different soil types and a layered or laminated structure is not evident.  
**Laminated** — Soil sample composed of alternating partings or seams of different soil type.

# UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS	SYM BOL	DESCRIPTION
Coarse-Grained Soils (more than 50% larger than No. 20 sieve)	GW	Well-Graded Gravel, Gravel-Sand Mixture
	GP	Poorly-Graded Gravel, Gravel-Sand Mixture
	GM	Silty Gravel, Gravel-Sand-Silt Mixture
	GC	Clayey-Gravel, Gravel-Sand-Clay Mixture
Sand and Gravelly Soils	SW	Well-Graded Sand, Gravelly Sand
	SP	Poorly-Graded Sand, Gravelly Sand
	SM	Silty Sand, Sand-Silt Mixture
	SC	Clayey Sand, Sand-Clay Mixture
Fine-Grained Soils (less than 50% larger than No. 20 sieve)	ML	Silt, Clayey Silt, Silty or Clayey Very Fine Sand, Slight Plasticity
	CL	Clay, Silty Clay, Silty Clay, Low to Medium Plasticity
	OL	Organic Silts or Silty Clays of Low Plasticity
	MH	Silt, Fine Sandy or Silty Soil with High Plasticity
	CH	Clay, High Plasticity
	OH	Organic Clay of Medium to High Plasticity
	PT	Peat, Humus, Swamp Soil

PLASTICITY CHART

Nonplastic  
Trace Plasticity  
Medium Plastic  
Highly Plastic

Cannot Roll into Ball  
Barely Roll into Ball  
Can Be Rolled into Ball  
No Rupture by Kneading

## VISUAL DESCRIPTION CRITERIA\*

TABLE 1: CRITERIA FOR DESCRIBING ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

TABLE 2: CRITERIA FOR DESCRIBING PARTICLE SHAPE

Description	Criteria
Flat	Particles with width/thickness X3
Elongated	Particles with length/width X3
Flat and Elongated	Particles meet criteria for both flat and elongated

TABLE 3: CRITERIA FOR DESCRIBING MOISTURE CONDITION

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

TABLE 4: CRITERIA FOR DESCRIBING REACTION WITH HCL

Description	Criteria
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

TABLE 5: CRITERIA FOR DESCRIBING CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

TABLE 6: CRITERIA FOR DESCRIBING DRY STRENGTH

Description	Criteria
None	The dry specimen crumbles into powder with mere pressure of handling
Low	The dry specimen crumbles into powder with some finger pressure
Medium	The dry specimen breaks into pieces or crumbles with considerable finger pressure
High	The dry specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and a hard surface
Very High	The dry specimen cannot be broken between the thumb and a hard surface

TABLE 7: CRITERIA FOR DESCRIBING DILATANCY

Description	Criteria
None	No visible change in the specimen
Slow	Water appears slowly on the surface of the specimen during shaking and does not disappear or disappears slowly upon squeezing
Rapid	Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing

TABLE 8: CRITERIA FOR DESCRIBING TOUGHNESS

Description	Criteria
Low	Only slight pressure is required to roll the thread near the plastic limit. The thread and the lump are weak and soft.
Medium	Medium pressure is required to roll the thread to near the plastic limit. The thread and the lump have medium stiffness
High	Considerable pressure is required to roll the thread to near the plastic limit. The thread and the lump have very high stiffness

TABLE 9: IDENTIFICATION OF INORGANIC FINE-GRAINED SOILS FROM MANUAL TESTS

Soil Symbol	Dry Strength	Dilatancy	Toughness
ML	None to low	Slow to rapid	Low or thread cannot be formed
CL	Medium to high	None to slow	Medium
MH	Low to medium	None to slow	Low to medium
CH	High to very high	None	High

\*NOTES: 1. Tables adapted from ASTM D 2488 "Description and Identification of Soils" (Visual-Manual Procedure)  
2. Tables 5, 7 and 11 incorporated into other information on this plate.

# **Geotechnical Calculations**

**EXHIBIT A-7.15**  
**Fairfax-Jersey Creek Flood Wall**  
**Pile Foundation Documentation**

## Appendix 4

### Fairfax-Jersey Creek Flood Wall Documentation

#### Development of the Ultimate Resistance of the Piles Below the Floodwall : Station 287+85 to 302+32.

The geometry of the existing flood wall can be found on O & MM Plate 38 of reference 11. The 17'-9" high stem of the floodwall sits 12'-6" back from the riverside face of the keyed riverside pile cap. The total width of the pile cap is 19'-3". In a cross section through the wall, three driven 15.5-inch diameter concrete piles support the pile cap. Nondestructive subsurface investigations concluded that one area of the floodwall has piles with a length of 19 feet below the pile cap.

The structural engineers needed ultimate geotechnical resistance of the driven piles. The subsurface investigation boring results were used to determine a cohesionless strength of the foundation sands adjacent to and below the piles. Both side resistance and end bearing was considered to determine the ultimate geotechnical resistance of the piles.

Side Resistance. The side resistance,  $Q_s$ , that develops adjacent to the piles is dependent on the construction placement procedure, the correlated soil strength, the length and diameter of the piles, and the pore pressures developed during loading along the piles.

The loading conditions include end of construction and varying river stages.

The side resistance calculated for a given increment of depth  $\Delta l$  below the pile cap was computed using the equation  $\Delta SR = f_{SR_z} * \Delta AREA$  where  $f_{SR_z}$  is the side resistance strength of the sand at depth  $z$  below the ground surface. The side resistance of the sand acts upon  $\Delta AREA$  of the pile.  $\Delta AREA$  is the perimeter of the piles for the length  $\Delta l$  or  $\Delta AREA = \pi * d * \Delta l$  with  $d$  being the diameter of the pile. The side resistance of the sand is dependent on the correlated effective friction angle of the sand at depth  $z$ . The lateral earth forces acting upon the incremental  $\Delta l$  and the effective pore pressure acts upon the incremental area of the pile,  $\Delta AREA$ . The pore pressure is calculated based on the river stage, or head ( $H$ ) acting on the flood wall and the sand foundation. The pore pressures were calculated using a simple two dimensional flow net based on the cross section of the flood wall and its foundation. The foundation is mostly sand with very little to no impervious blanket. The flow net provided equipotential drop lines from full head. These lines were used to determine the pore pressures along the piles for various river stages.

The magnitude of the effective pressure acting along the sides of the piles is sensitive to the placement techniques used to construct the piles. The piles were driven. The magnitude of the compressional coefficient of pressure used for analysis,  $K_c$ , was set to 1.0. EM 1110-2-2906 allows 1.0 to 2.0. The effective lateral pressure was

reduced by the pore pressure developed for each river stage used in the analysis. The effective lateral pressure was multiplied by the effective friction angle for sand acting on a concrete surface,  $\delta'$ . The values suggested in EM 1110-2-2906 ranged from  $0.9\phi'$  to  $1.0\phi'$ . The values used for the analysis are more conservative than the EM and are consistent with that suggested by Reese. The value  $\delta'$  used was  $\phi' - 5^\circ$ . DSR

The incremental side resistance,  $\Delta SR$ , was computed as  $f_s * \Delta AREA$  or  $[(\sigma_{tz} - \mu_{wz}) * K_c] * [\tan(\delta') * \pi * d * (\Delta l_z)]$ .

Where  $\sigma_{tz}$  is the total pressure at depth  $z$ ;

and  $\mu_{wz}$  is the pore pressure at depth  $z$ ;

and  $\Delta l_z$  is the increment length of pile at depth  $z$ .

The spreadsheet calculates the resistance for each increment and adds the increments as the depth  $z$  increases. A total ultimate capacity is provided at the bottom for the total length of the pile.

End Bearing Resistance. The end bearing pressure acting on the area below the pile provides the end bearing capacity. The end bearing capacity is dependent on the effective friction angle of the sand, the effective overburden pressures below the pile and the cross sectional area of the bottom of the pile.

$$Q_t = q_{br} * A_t \text{ below pile for depth } z$$

Where  $q_{br} = \sigma_v' * N_q$ .  $N_q$  is the expected value of the bearing capacity factor dependent of the overburden

surcharge loading (depth of soil) and the area of the bottom of the pile,  $A_t$ , is  $\pi * (d/2)^2$ , with  $d$  being the diameter of the pile.

The bearing capacity factor used the mean values of Figure 4-4. The equation used in the spreadsheet was  $N_q = 0.8158 * e^{(0.1165\phi')}$ . The values used for  $\sigma_v'$  were based on the total overburden pressure less the pore pressures,  $\sigma_{vzt} - \mu_{wz}$ , developed for each river stage. The pore pressure was determined using the flow net.

Ultimate Pile Capacity. The ultimate capacity of the pile is determined by adding the side resistance and the end bearing resistance.

Results of the analysis. The results are provided for the landside, middle and riverside piles. A series of results are provided for the mean, mean - 1 standard deviation, mean + 1 standard deviation and for the one third  $\phi'$  strength selection of the friction angle at the base of the piles. The friction angle along the sides of the piles varied with depth dependent on the blow counts taken. These values were also adjusted to provide mean, mean-1, mean+1 and one third strength values with depth. The results were provided for height of water on the flood wall of 8, 13 and 18 feet.

Per EM 1110-2-2906 ; Dated 15 Jan 1991  
P. 4-11

$$Q_{ult} = Q_s + Q_t$$

$Q_s$  = shaft resistance

$Q_t$  = tip resistance

$$Q_s = f_s A_s \quad ; \quad f_s = \text{average unit skin resistance} \\ \Delta SR_z$$

$$f_s = K \sigma_v' \tan \delta$$

Use  $f_s$  calculated to a maximum value at some critical depth

$$D_c = 10 B \quad - \text{loose soils}$$

$$D_c = 15 B \quad - \text{med dense soils}$$

$$D_c = 20 B \quad - \text{dense soils}$$

$$\sigma_v' = \gamma' D \quad \text{for } D < D_c$$

$$\sigma_v' = \gamma' D_c \quad \text{for } D > D_c$$

$K$  = lateral earth pressure coefficient

from Table 4-3 p. 4-12

Value of  $\delta \rightarrow$  Concrete Pile  $\delta$  varies from  $0.9\phi$  to  $1.0\phi$

Per Reese use  $\phi = 5^\circ = \delta$  more conservative than  $0.9\phi$

Values of  $K$  Table 4-4

For Sand  $K_c \sim 1.0$  to  $2.0$  ;  $K_t \sim 0.5$  to  $0.7$   
Compressional Tensional

### END BEARING

DESIGN PURPOSES USE LINEAR INCREASE TO  $D_c$  THEN USE  
CONSTANT BELOW THAT

$$q_{br} = \sigma'_v N_g \quad N_g \text{ from Fig 4-4}$$

$$\begin{aligned} \text{Limit } \sigma'_v &= \gamma' D \text{ for } D < D_c \\ &\& \sigma'_v = \gamma' D_c \text{ for } D > D_c \end{aligned}$$

Using  $D_c = 15B$  where  $B = 15.5'$

$$D_c = \left(\frac{15.5}{12}\right) * 15 = 19.4' \text{ Below Pile Cap}$$

Pile cap base thickness  $\approx 3'$

Limiting depth below ground  $D_c = 19 + 3 = 22'$  (Design)

The limiting depth control was ignored for two reasons:

- 1) Depth of Piles =  $19' < 22'$
- 2) It is a requirement for DESIGN, we are studying the existing conditions.

10/5/2004  
Loehr EC-GD

Ref ETL 1110-2-556  
dated 28 May 1999

Use Variation of  $\phi'$  parameter of 10%  
Std Dev =  $\sigma_d = V * E(\phi')$

$$\begin{aligned}\text{Vertical Capacity of pile} &= \text{Side Resist} + \text{End Bearing} \\ &= (\alpha * \text{Perimeter} * h) + \sigma'_v N_g \\ &= [(\sigma'_h + \tan \phi')(\pi d h)] + [\sigma'_v * N_g]\end{aligned}$$

## Phi vs Nq Driven piles

$$N_q = 0.8158e^{(0.1165 * \phi)}$$

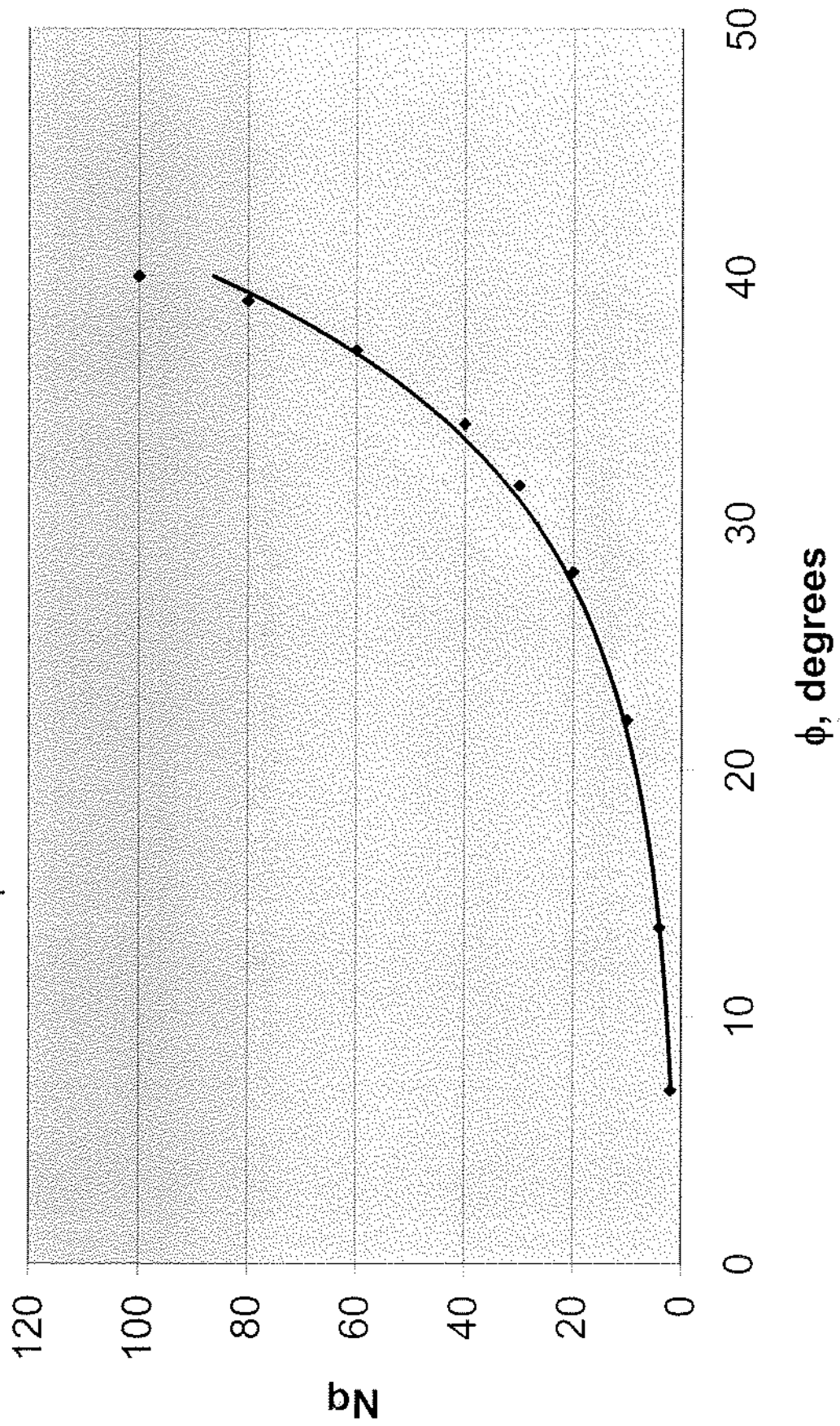


Table 1 provides a summary of typical reported values for the coefficients of variation of commonly encountered geotechnical parameters. More detailed comment regarding the observed variability of relevant parameters is provided in the subsequent sections.

Table 1 Coefficients of Variation for Geotechnical Parameters		
Parameter	Coefficient of Variation, percent	Reference
Unit weight	3 4 to 8	Hammitt (1966), cited by Harr (1987) assumed by Shannon and Wilson, Inc., and Wolff (1994)
Drained strength of sand $\phi'$	3.7 to 9.3 12	Direct shear tests, Mississippi River Lock and Dam No. 2, Shannon and Wilson, Inc., and Wolff (1994) Schultze (1972), cited by Harr (1987)
Drained strength of clay $\phi'$	7.5 to 10.1	S tests on compacted clay at Cannon Dam, Wolff (1985)
Undrained strength of clay $s_u$	40 30 to 40 11 to 45	Fredlund and Dahlman (1972) cited by Harr (1987) Assumed by Shannon and Wilson, Inc., and Wolff (1994) Q tests on compacted clay at Cannon Dam, Wolff (1985)
Strength-to-effective stress ratio $s_u / \sigma'_v$	31	Clay at Mississippi River Lock and Dam No. 2, Shannon and Wilson, Inc., and Wolff (1994)
Coefficient of permeability $k$	90	For saturated soils, Nielson, Biggar, and Erh (1973) cited by Harr (1987)
Permeability of top blanket clay $k_b$	20 to 30	Derived from assumed distribution, Shannon and Wilson, Inc., and Wolff (1994)
Permeability of foundation sands $k_f$	20 to 30	For average permeability over thickness of aquifer, Shannon and Wilson, Inc., and Wolff (1994)
Permeability ratio $k_f / k_b$	40	Derived using 30% for $k_f$ and $k_b$ ; see Annex B
Permeability of embankment sand	30	Assumed by Shannon and Wilson, Inc., and Wolff (1994)

$$V = \frac{\sigma}{E(c)}$$

$$\sigma = V * E(c)$$

$$\text{Use } 10\% = V$$

## Unit Weight of Soil Materials

The coefficient of variation of the unit weight of soil material is usually on the order of 3 to 8 percent. In slope stability problems, uncertainty in unit weight usually contributes little to the overall uncertainty, which is dominated by soil strength. For stability problems, it can usually be taken as a deterministic variable in order to reduce the number of random variables and simplify calculations. It

15 Jan 91

where

$c_a$  = adhesion between the clay and the pile

$\alpha$  = adhesion factor

$c$  = undrained shear strength of the clay from a Q test

The values of  $\alpha$  as a function of the undrained shear are given in Figure 4-5a.

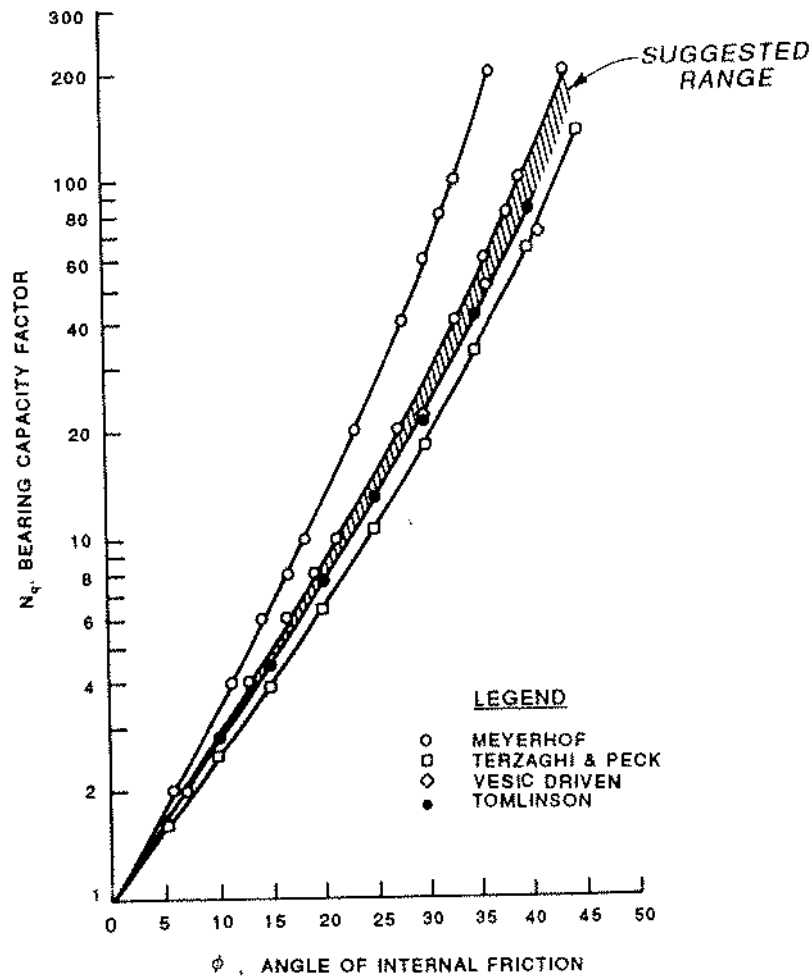


Figure 4-4. Bearing capacity factor

An alternate procedure developed by Semple and Rigden (Item 56) to obtain values of  $\alpha$  which is especially applicable for very long piles is given in Figure 4-5b where:

$$\alpha = \alpha_1 \alpha_2$$

and

$$f_s = \alpha c$$

**EXHIBIT A-7.16**  
**Fairfax-Jersey Creek Flood Wall**  
**Flownet**

LANDSIDE

RIVERSIDE

Using LOSD Ref Datum  
h = 16' above LOSD ref datum

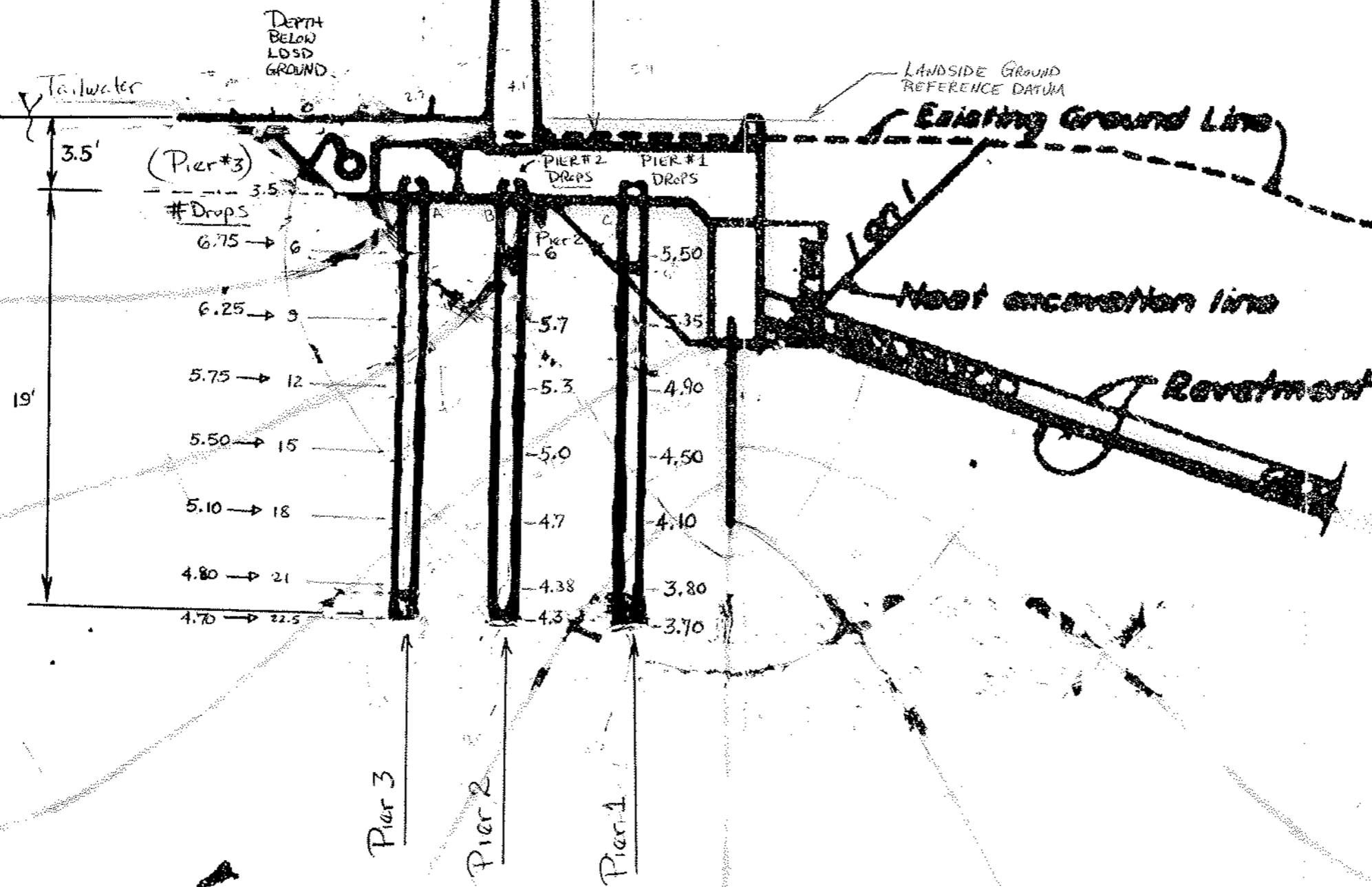
$$\Delta h = 2.25 \text{ ft for } 18'$$

$$N_e = 8$$

18' = Full Head  
(16.5' Head Above LOSD Ref Datum)

$$\text{Net head} = h - \Delta h \left( \frac{N_e}{\# \text{ Drops}} \right)$$

Full Head  
Line of Creep  
Flow Lines  
Equipotential Head Lines



- Flow Net Assumptions
- DRAIN INEFFECTIVE
  - CUTOFF EFFECTIVE

SCALE 1" = 6'

$\bar{p}$  = effective overburden pressure at the point in question, and

$\delta$  = the friction angle between the soil and the pile wall.

A value of  $K$  of 0.8 was recommended for open-ended pipe piles, that are driven unplugged, for loadings in both tension and compression. A value of  $K$  of 1.0 was recommended for full displacement piles. In the absence of data on  $\delta$ , Table 3.1 was recommended as guidelines only for siliceous soil.

Equation 3.9 implies that the value of  $f$  increases without limit; however, Table 3.1 presents guidelines for limiting values.

TABLE 3.1. Guideline for Side Friction in Siliceous Soil  
 $\delta = 3-5^\circ$  (Assumed)

Soil	$\delta$ , degrees	Limiting $f$ , Kips/ft <sup>2</sup> (kPa)
Very loose to medium, sand to silt	15	1.0 (47.8)
Loose to dense, sand to silt	20 $\leftarrow \delta = 22^\circ \rightarrow$	1.4 (67.0)
Medium to dense, sand to sand-silt	25 $\leftarrow \delta = 25^\circ \rightarrow$	1.7 (83.1)
Dense to very dense, sand to sand-silt	30 $\leftarrow \delta = 28^\circ \rightarrow$	2.0 (95.5)
Dense to very dense, gravel to sand	35	2.4 (114.8)

### 3.3.5 End Bearing in Cohesionless Soil

For end bearing in cohesionless soils, API recommends the following.

$$q = \bar{p}_o N_q \quad (3.10)$$

where

$\bar{p}$  = effective overburden pressure at pile tip, and

$N_q$  = bearing capacity factor.

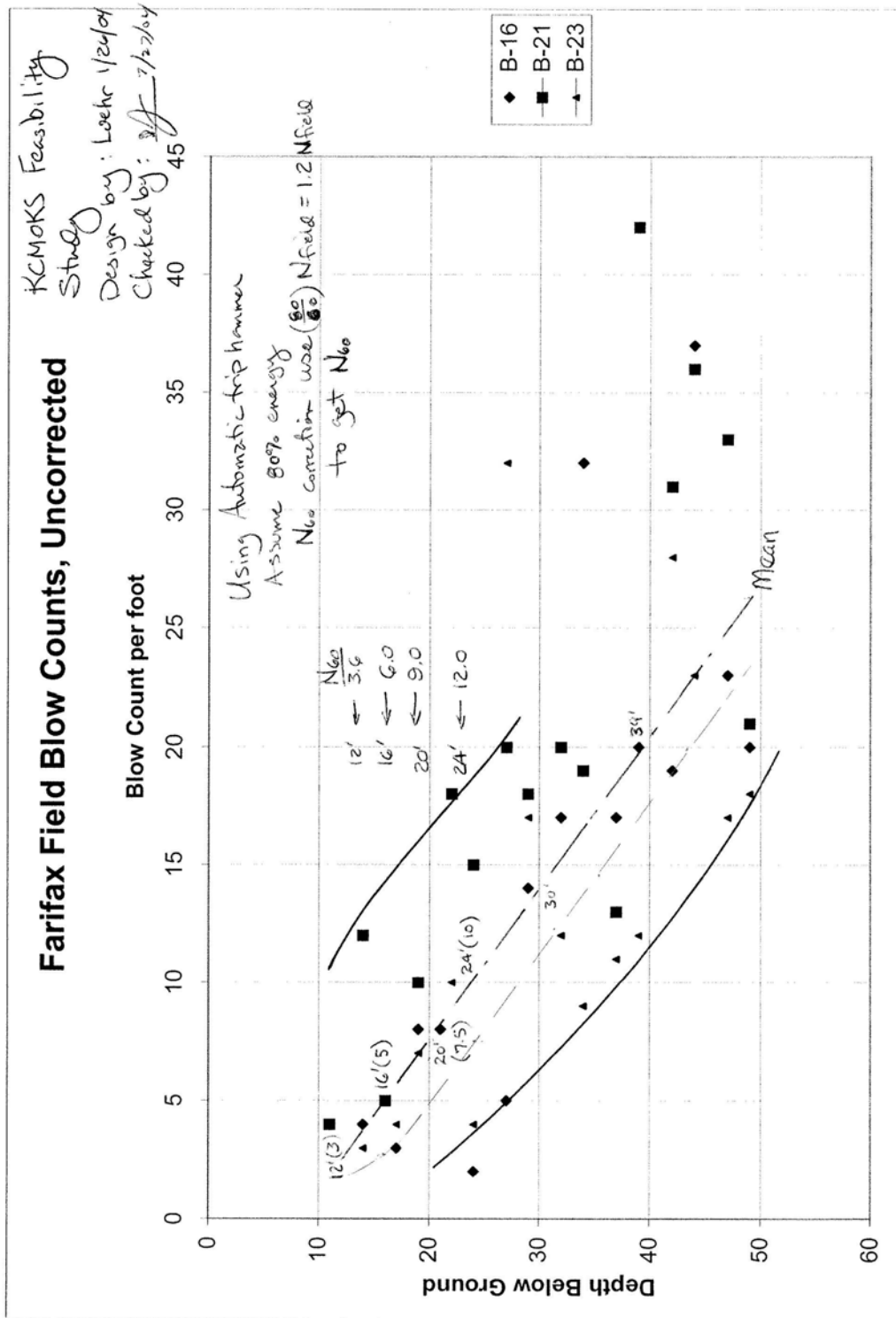
Table 3.2 was recommended as a guideline only for siliceous soil.

TABLE 3.2. Guideline for Tip Resistance in Siliceous Soil

Soil	$N_q$	Limiting $q$ , kips/ft <sup>2</sup> (MPa)
Very loose to medium, sand silt	8	40 (1.9)
Loose to dense, sand to silt	12 $\leftarrow \delta = 22^\circ \rightarrow$	60 (2.9)
Medium to dense, sand to sand-silt	20 $\leftarrow \delta = 25^\circ \rightarrow$	100 (4.8)
Dense to very dense sand to sand-silt	40 $\leftarrow \delta = 28^\circ \rightarrow$	200 (9.6)
Dense to very dense, gravel to sand	50	250 (12.0)

The API publication points out that many soils do not fit the description of those in the tables and that the design parameters are not suitable for these soils. Examples are loose silts, soils containing large amounts of mica or volcanic grains, and calcareous sands. These latter soils are known to have substantially lower design parameters.

Drilled and grouted piles may have higher capacities than driven piles in calcareous soils.



# EXHIBIT A-7.19

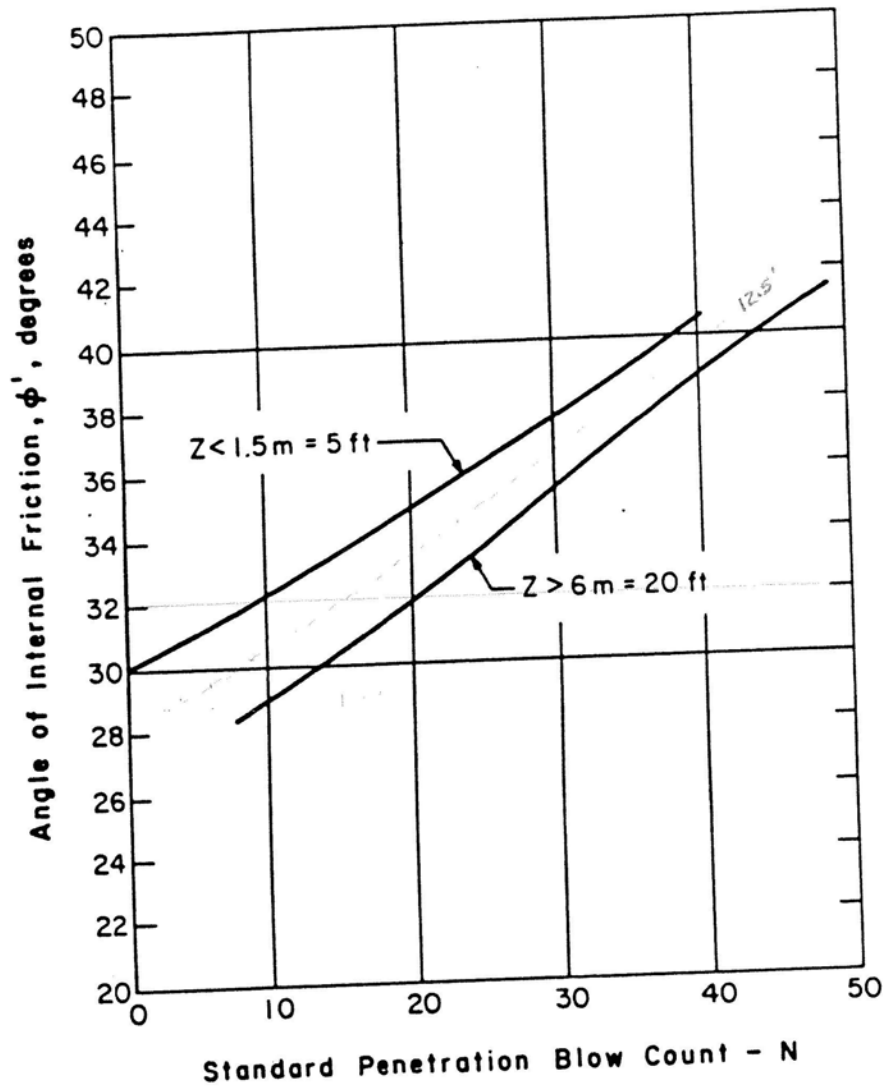
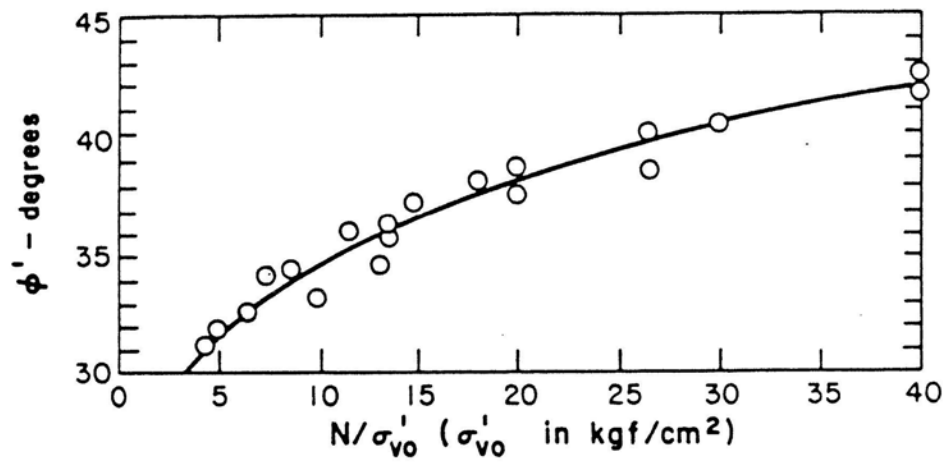


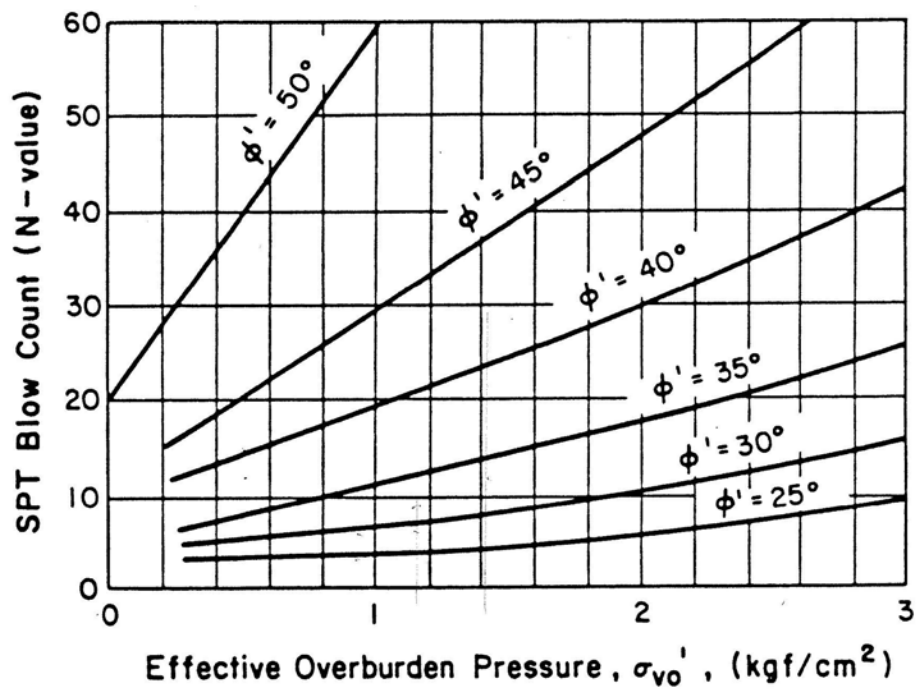
Fig.13 Relationship between Angle of Internal Friction of Cohesionless Soils and SPT Blow Count, (After Sowers, 1979).

Ref: "Shear Strength Correlations for Geotechnical Engineering", Duncan, Horz, Yang, August, 1989

# EXHIBIT A-7.20



(a) Relation between  $\phi'$  and  $N/\sigma'_{vo}$ , (After Parry, 1977).



(b) Relationship among  $\phi'$ , SPT-N and  $\sigma'_{vo}$ , (After DeMello, 1971 and Schmertmann, 1975).

Fig.14 Relationship among Overburden Pressure,  $\sigma'_{vo}$ , Blow Count, N, and Peak Friction Angle,  $\phi'$ , for Sands.



**EXHIBIT A-7.21 (Continued)**[illegible]

NOTES:

1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landslide ground surface to determine the total pressure head.

Notes : Nqp from Figure 4-4 of EM 1110-2-25 with equation developed a  $N_q = 0.8158e(0.1165*\phi')$   
 $q_b = \sigma'v' * N_{qp}$

Side resistance Along Pier = Sum ( $\tau_{dl}$  \* Area<sub>dl</sub>) , dl = incremental pile length

Example : at  $l = 9$  feet; Side resistance =  $\left[ \frac{(0+151)}{2} * (6-3.5) * 4.058 * \tan 22 \right] + \left[ \frac{(151+326)}{2} * (9-6) * 4.058 * \tan 22 \right] = 1483$

### Pressure Along Pier Due to Equipotential Drop Using Flow net

**Net Head Above Landside Existing Ground Surface:**

$$\text{Net Head} = h - \Delta h * (\# \text{Drops})$$

$h$  = head above landside ground surface due to the river level

$$\Delta h = \text{head drop for one equipotential drop} = h / N$$

Ne = total equipotential drops

rops : total increment of equipo

along the pier.

9-11

**Example :** Pier 3, 6 feet below the top of the landside ground surface for 13 feet of water on the floodwall  $h = 13' - 1.5' = 11.5'$

$N_e$  from the flow net = 8

$$\Delta h = h/N_x = 11.5/8 = 1.4375$$

# Drops = 5.5

# Drops = 5.5  
d above landsi

Total pressure at 6 foot : P

Total pressure at 0 feet,  $P_0$

$$P_6 = (\text{Net head} - \text{elevation head}) * \text{Unit weight of water}$$
$$P_6 = (3.60 - (-6)) * 62.4$$
 $P_c = 599 \text{ nsf}$ 

16 000 per

EXHIBIT A-7.21 (Continued)

Fairfax Floodwal Study Feasibility							Using unit weight sand = 120 pcf moist							Side resistance = f(phi and effective pressure)										Using 15.5-inch diameter piers													
Riverside Pile			Geotechnolgy Results				Water depth not encountered							Designer : Loehr 12/10/2004							$\tau = ( \sigma' - \mu_u ) * \tan \delta'$					Perimeter = 2 * pi * r =					4.058 feet						
Water at TOW Head =18'			Automatic Trip Hammer				Use 80%							Checker : Kuzniakowski 12/2004							Using $\gamma_{sat} = 125$ pcf					Using Ko = 1 - sin $\phi'$					Area = pi * radius^2					1.310 sq ft	
							Full Head Conditions =====> Head = 18 feet																														
Uncorrected Blowcount	Material	Uncorrected Blowcount	Material	Uncorrected Blowcount	Material		Selected Mean Value	Conversion Automatic Trip Hammer	Sowers 1979 Phi Angle Fig 13 Degrees	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi - 5$ degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Riverside Pile Psf	Effective Overburden Pressure $\sigma' v'$ , Psf	Effective Overburden Pressure $\sigma' h'$ , Psf	Ultimate Shear along Length $\tau$ , psf	Side Resistance Along Pier Length pounds	$N_{90}$ End Bearing coefficient dimensionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds													
B-16							B-21							B-23							Driven Piers Use K =1																
0											Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0																	
																			0	0																	
												27	22	3	375	496	0	0	0	0																	
														3.5		5.6																					
5												27	22	6	750	696	54	54	10.88	27	18.95	1021	1337	1365													
																5.5																					
												27	22	9	1125	903	222	222	55.79	679	18.95	4214	5522	6228													
10																5.35			116.07	1413																	
11			4	Silty Sand																2119	18.95	6675	8747	10867													
12							3	3.6	29	25	0.703	27	22	12	1500	1148	352	352	4.9																		
14	4	Silty Sand	12	Silty Sand	3	Silty Clay													174.13	2120																	
																				4239	21.29	10403	13632	17871													
16												28	23	15	1875	1386	489	489	4.5																		
17	3	Silty Sand		5	Silty Sand			5	6	29	29	0.937	29	24																							
												29.5	24.5						247.86	3017																	
												30	25	18	2250	1625	625	625		7257	26.88	16797	22010	29267													
19	8	Silty Sand	10	Silt	7												4.1																				
20																																					
21	8	Silty Sand				Sand		7.5	9	28.5	32	1.171	30.25	25.25					328.40	3998																	
22															2625	1851	774	774		11254	27.84	21547	28235	39489													
																			386.79	2354																	
24	2	Silty Sand	15	Sand	4	Sand		10	12	29.5	33	1.405	31.25	26.25	24	3000	3.7			13609	28.49	24364	31926	45534													
27	5	Sand	20	Sand		Sand									27	3375				13609 Pounds Sum of Side Resistance 19' length pier			31926 Pounds Sum of Side Resistance 19' length pier	45534 Pounds Sum of Side Resistance 19' length pier	For Ultimate Displacement expect 10% of pier diameter or 1.8 inches												

EXHIBIT A-7.22

Fairfax Floodwall Study Feasibility						Using unit weight sand = 120 pcf moist						Side resistance = f(phi and effective pressure)						Using 15.5-inch diameter piers					
Middle Pier						Water depth not encountered						τ = ( σ t - μ <sub>u</sub> ) * tan δ '						Perimeter = 2 * pi * r =					
Head = 8						Automatic Trip Hammer						Using γ <sub>sat</sub> = 125 pcf						Area = pi * radius*2					
Use 80%																							

EXHIBIT A-7.22 (Continued)

Fairfax Floodwall Study Feasibility						Using unit weight sand = 120 pcf moist						Side resistance = f(phi and effective pressure)						Using 15.5-inch diameter piers																															
Middle Pile			Geotechnology Results			Water depth not encountered			Designer : Loehr 12/10/2004			$\tau = ( \sigma t - \mu_{so} ) * \tan \delta ^{\circ}$						Perimeter = 2 * pi * r =			4.058 feet																												
Water at TOW Head =13'			Automatic Trip Hammer			Use 80%			Checker : Kuzniakowski 12/2004			Using $\gamma_{sat}$ = 125 pcf			Using Ko = 1 - sin $\phi ^{\circ}$			Area = pi * radius^2			1.310 sq ft																												
Uncorrected Blowcount		Material		Uncorrected Blowcount		Material		Uncorrected Blowcount		Material		Selected Mean Value		Conversion Automatic Trip Hammer Use 1.2		Sowers 1979 Phi Angle Fig 13 Degrees		DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees		Effective Overburden Pressure (kgf / sq cm)		Average of (1) AND (2) degrees		$\delta = \phi - 5$ degrees		Depth		Total Pressure Along Pier Psf		Pressure from Flow Net along Middle Pile Psf		Effective Overburden Pressure $\sigma v'$ , Psf		Effective Overburden Pressure $\sigma h'$ , Psf		Ultimate Shear along Length $\tau_c$ , psf		Side Resistance Along Pier Length pounds		N <sub>sp</sub> End Bearing coefficient dimensionless		End Bearing Pressure qb, psf		End Bearing Resistance Qb, Pounds		Ultimate Side and End Bearing Pounds			
B-16				B-21				B-23																																									
Depth																																																	
																								Note : 1 Tsf = 0.976 kgf /																									
0																						27		22		0		0		0		0		0															
																						27		22		3		375		349		26		26				0											
																										3.5				6.2																			
5																										6		750		554		196		196				45		1361									
10																																																	
11				4		Silty Sand																																											
12								3		3.6		29		25		0.703		27		22		12				1500		991		509		509				175		2130		3947		18.95		9647		12641		16587	
14		4		Silty Sand		12		Silty Sand		3		Silty Clay																																					
16						5		Silty Sand										28		23		15				1875		1205		670		670				244		2972		6919		21.29		14265		18692		25611	
17		3		Silty Sand								Silty Clay						29		24										5																			
19		8		Silty Sand		10		Silt		7								29.5		24.5						2250		1419		831		831				334		4067		10986		26.88		22332		29263		40249	
20																																																	
21		8		Silty Sand								Sand		7.5		9		28.5		32		1.171		30.25		25.25										428		5208											
																								30.3		25.3		21		2625		1633		992		992		491		16194		27.84		27605		36173		52367	
22.5						18		Sand		10		Sand												30.5		25.5		22.5		2812.5		1736		1077		1077				19183		28.49		30676		40196		59379	
24		2		Silty Sand		15		Sand		4		Sand		10		12		29.5		33		1.405		31.25		26.25		24										19183 Pounds Sum of Side Resistance 19' length pier						40196 Pounds Sum of Side Resistance 19' length pier		59379 Pounds Sum of Side Resistance 19' length pier		For Ultimate Displacement expect 10% of pier diameter or 1.8 inches	
27		5		Sand		20		Sand		32		Sand																																					

**EXHIBIT A-7.22 (Continued)**

Fairfax Floodwall Study Feasibility						Using unit weight sand = 120 pcf moist						Side resistance = f(phi and effective pressure)						Using 15.5-inch diameter piers						
Middle Pile						Water depth not encountered			Designer : Loehr 12/10/2004			τ = ( σ1 - μu ) * tan δ '			Perimeter = 2 * pi * r =			4.058 feet						
			Geotechnology Results						Checker : Kuzniakowski 12/2004			Using γsat = 125 pcf			Using Ko = 1 - sin φ '			Area = pi * radius^2			1.310 sq ft			
Water at TOW Head =18'			Automatic Trip Hammer			Use 80%																		
Uncorrected Blowcount	Material	Uncorrected Blowcount	Material	Uncorrected Blowcount	Material	Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	δ = φ - 5 degrees	Depth	Total Pressure Along Pier Psf	Pressure from Flow Net along Middle Pile Psf	Effective Overburden Pressure σ v ' , Psf	Effective Overburden Pressure σ h ' , Psf	Ultimate Shear along Length τ , psf	Side Resistance Along Pier Length pounds	Nsp End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds	
Depth						Full Head Conditions =====> Head = 18 feet																		
						Driven Piers Use K =1																		
0										Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0					
																		0	0					
											27	22	3	375	419	0	0	0	0					
													3.5	6.2										
5											27	22	6	750	632	118	118	23.88	60	18.95	2240	2935	2995	
															6									
											27	22	9	1125	858	267	267	77.89	948	1008	5068	6640	7648	
10																								
11			4	Silty Sand														135.57	1650					
12						3	3.6	29	25	0.703	27	22	12	1500	1096	404	404	5.3	2658	18.95	7651	10026	12684	
14	4	Silty Sand	12	Silty Sand	3	Silty Clay												198.12	2412					
											28	23	15	1875	1322	553	553		5070	21.29	11773	15427	20497	
16											29	24												
17	3	Silty Sand		Silty Sand	4	Silty Clay	5	6	29	29	0.937	29						5						
											29.5	24.5												
											30	25	18	2250	1548	702	702		279.38	3401				
19	8	Silty Sand																4.7		8471	26.88	18873	24730	33201
20																								
21	8	Silty Sand									30.25	25.25	21	2625	1774	851	851		364.65	4439				
22											30.3	25.3							12910	27.84	23697	31052	43962	
											30.5	25.5							4.4	2577				
											30.5	25.5	22.5	2812.5	1880	932	932		423.45	2577				
24	2	Silty Sand	15	Sand	4	Sand	10	12	29.5	33	1.405	31.25	26.25	24	3000	4.3								
27	5	Sand	20	Sand	32	Sand								27	3375									

NOTES:

1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head.

**Notes :**

Nqp from Figure 4-4 of EM 1110-2-291 with equation developed as:  $N_q = 0.8158e(0.1165 \cdot \phi')$   
 $q_b = \sigma_v' \cdot N_{qp}$

Side resistance Along Pier = Sum ( $\tau_{dl}$  \* Area<sub>dl</sub>) , dl = incremental pile length

Example : at  $l = 9$  feet; Side resistance =  $\left[ \frac{(0+118)}{2} * (6-3.5) * 4.058 * \tan 22 \right] + \left[ \frac{(118+267)}{2} * (9-6) * 4.058 * \tan 22 \right] = 1008$

### Pressure Along Pier Due to Equipotential Drop Using Flow net

### Net Head Above Landside Existing Ground Surface:

$$\text{Net Head} = h - \Delta h^* (\# \text{Drops})$$

$h$  = head above landside ground surface due to the river leve

$$\Delta h = \text{head drop for one equipotential drop} = h / N_e$$

Ne = total equipotential drops

rops : total increment of equipment

# Drops : total increment of equipotential drop to the location in the foundation along the pier.

### Example

Pier 3, 6 feet below the top of the landside ground surface  
for 13 feet of water on the floodwall  $h = 18' - 1.5' = 16.5'$

$N_e$  from the flow net = 8

$$\Delta h = h/N_o = 16.5/8 = 2.06$$

# Drops = 6

Net Head above landside ground surface =  $16.5 - (2.06)^6 = 4.14$

Total pressure at 6 feet :  $P_6$

$$P_o = (\text{Net head} - \text{elevation head}) * \text{Unit weight of water}$$

4.14 (6) \* 62.4

D. 333-16

 $\tau_6 = 0.52 \text{ ps}$

EXHIBIT A-7.23

Fairfax Floodwall Study Feasibility						Using unit weight sand = 120 pcf moist						Side resistance = f(phi and effective pressure)						Using 15.5-inch diameter piers					
Landside Pile			Geotechnology Results			Water depth not encountered			Designer : Loehr 12/10/2004			$\tau = ( \sigma' - \mu_{w0} ) * \tan \delta'$			Perimeter = 2 * pi * r =			4.058 feet					
Head = 8			Automatic Trip Hammer			Use 80%			Checker : Kuzniakowski 12/2004			Using $\gamma_{sat}$ = 125 pcf			Using Ko = 1 Driven piers			Area = pi * radius^2			1.310 sq ft		
Uncorrected Blow count	Material	Uncorrected Blow count	Material	Uncorrected Blow count	Material	Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi - 5$ degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Landside Pile Psf	Effective Overburden Pressure $\sigma' v'$ , Psf	Effective Overburden Pressure $\sigma' h'$ , Psf	Ultimate Shear along Length $\tau$ , psf	Total Side Rst Along Pier Length pounds	Nqp End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds
B-16		B-21		B-23																			
Depth																							
0										Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0	18.95	0	0	0
											27	22	3	375	238	137	137	0	0	18.95	2598	0	0
5																							
10																							
11			4	Silty Sand																			
12						3	3.6	29	25	0.703	27	22	12	1500	863	637	637	224.60	5532	18.95	12075	15822	21354
14	4	Silty Sand	12	Silty Sand	3	Silty Clay																	
16			5	Silty Sand			5	6	29	0.937	28	23	15	1875	1063	812	812		9186	21.29	17296	22664	31850
17	3	Silty Sand			4	Silty Clay					29.5	24.5											
19	8	Silty Sand	10	Silt	7						30	25	18	2250	1270	980	980		14042	26.88	26337	34511	48553
20																							
21	8	Silty Sand					7.5	9	28.5	1.171	30.25	25.25											
22.5											30.3	25.3	21	2625	1473	1152	1152	500.51	20135	27.84	32078	42034	62169
24	2	Silty Sand	15	Sand	4	Sand	10	12	29.5	1.405	31.25	26.25		3000									
27	5	Sand	20	Sand	32	Sand																	
29	14	Sand	18	Sand	17	Sand																	
30							14	16.8	31	1.757	33												
32	17	Sand	20	Sand	12	Sand																	
34	32	Sand	19	Sand	9	Sand																	
35																							
37	17	Sand	13	Sand	11	Sand																	
39	20	Sand	42	Sand	12	Sand	20	24	33	2.284	33												
42	19	Sand	31	Sand	28	Sand																	
44	37	Sand	36	Sand	23	Sand																	
46							25	30	35	2.694	36												
47	23	Sand	33	Sand	17	Sand																	
49	20	Sand	21	Sand	18	Sand																	

EXHIBIT A-7.23 (Continued)

Fairfax Floodwall Study Feasibility							Using unit weight sand = 120 pcf moist							Side resistance = f(phi and effective pressure)							Using 15.5-inch diameter piers													
Landside Pile			Geotechnology Results				Water depth not encountered				Designer : Loehr 12/10/2004				$\tau = ( \sigma' - \mu_u ) * \tan \delta'$							Perimeter = 2 * pi * r =				4.058 feet								
Water at TOW Head =13'			Automatic Trip Hammer				Use 80%				Checker : Kuzniakowski 12/2004				Using $\gamma_{sat}$ = 125 pcf							Using $K_o$ = 1 - sin $\phi'$							Area = pi * radius^2				1.310 sq ft	
Uncorrected Blowcount	Material	Uncorrected Blowcount	Material	Uncorrected Blowcount	Material		Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees	DeMello 1971 1975 Phi Angle Fig 14 Degrees	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi - 5$ degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Landside Pile Psf	Effective Overburden Pressure $\sigma' v'$ , Psf	Effective Overburden Pressure $\sigma' h'$ , Psf	Ultimate Shear along Length $\tau$ , psf	Side Resistance Along Pier Length pounds	$N_{90}$ End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds										
Depth														Driven Piers Use K =1																				
0											Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0														
												27	22	3	375	277	98	98		0														
														3.5																				
5									Top of Pile : 3.5'					6	750	487	263	263	73	741														
												27	22							741														
												27	22	9	1125	719	406	406	135	1647														
10																				2388														
11			4	Silty Sand															6.25		18.95	7703	10093	12482										
12							3	3.6	29	25	0.703	27	22	12	1500	951	549	549	193	2351														
																				4739														
14	4	Silty Sand	12	Silty Sand	3	Silty Clay													5.75		18.95	10412	13643	18382										
16			5	Silty Sand			5	6	29	29	0.937	28	23	15	1875	1160	715	715	262	3187														
17	3	Silty Sand			4	Silty Clay						29	24					5.5		7926		21.29	15220	19943	27869									
												29.5	24.5																					
												30	25	18	2250	1383	867	867	352	4286														
19	8	Silty Sand	10	Silt	7																													
20																																		
21	8	Silty Sand				Sand	7.5	9	28.5	32	1.171	30.25	25.25																					
												30.3	25.3	21	2625	1597	1028	1028	445	5413														
22.5			18	Sand	10	Sand						30.5	25.5	22.5	2812.5	1700	1112	1112	508	17625	27.84	28604	37482	55107										
24	2	Silty Sand	15	Sand	4	Sand	10	12	29.5	33	1.405	31.25	26.25	24																				
27	5	Sand	20	Sand	32	Sand								27																				

**EXHIBIT A-7.23 (Continued)**

Fairfax Floodwall Study Feasibility						Using unit weight sand = 120 pcf moist						Side resistance = f(phi and effective pressure)					Using 15.5-inch diameter piers																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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Water at TOW Head =18'		Geotechnology Results						Checker : Kuzniakowski 12/2004				Using $\gamma_{sat} = 125$ pcf					Using $K_o = 1 - \sin \phi'$					Area = pi * radius^2					1.310 sq ft																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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NOTES:

1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head.  
where pressure head = Net head above landside ground surface - depth below landside ground surface.

Notes :  $N_q$  from Figure 4-4 of EM 1110-2-29 (with equation developed as  $N_q = 0.8158e(0.1165 \cdot \phi')$ )  
 $q_b = \sigma' \cdot N_q$

Side resistance Along Pier = Sum ( $\tau_{dl} \cdot \text{Area}_{dl}$ ) , dl = incremental pile length

Example : at  $l = 9$  feet; Side resistance =  $[(59+215)/2 * (6-3.5)*4.058*\tan 22)] + [(215+338)/2 * (9-6)*4.058*\tan 22) = 1498$

### Pressure Along Pier Due to Equipotential Drop Using Flow net

### Net Head Above Landside Existing Ground Surface.

Net Head =  $h - \Delta h^* (\# \text{ Drops})$   
 $h$  = head above landside ground surface due to the river level  
 $\Delta h$  = head drop for one equipotential drop =  $h / N_e$   
 $N_e$  = total equipotential drops  
 $\# \text{ Drop}$  : total increment of equipotential drop to the location in the four along the pier.

**Example :**

Pier 3, 6 feet below the top of the landslide ground surface for 13 feet of water on the floodwall  $h = 18' - 1.5' = 16.5'$

$N_b$  from the flow net = 8

$\Delta h = h/N_b = 16.5/8 = 2.06$

# Drops = 6.75

Net Head above landslide ground surface =  $16.5 - (2.06)6.75 = 2.58'$

Total pressure at 6 feet ;  $P_6$

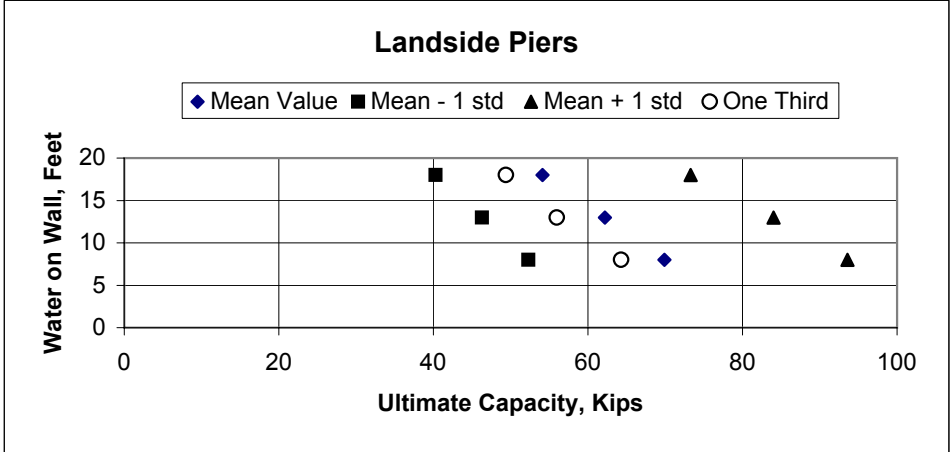
$P_6 = (\text{Net head} - \text{elevation head}) * \text{Unit weight of water}$

$P_6 = (2.58 - (-6)) * 62.4$

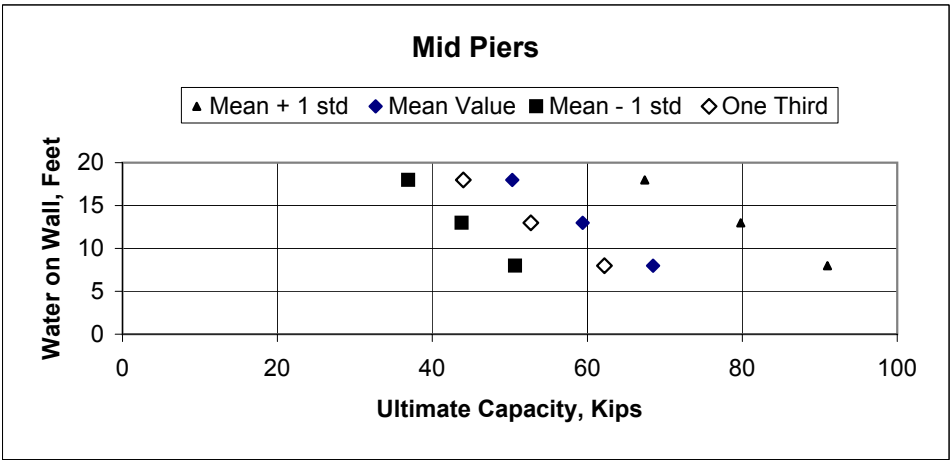
$P_6 = 535 \text{ psf}$

EXHIBIT A-7.24

Phi angle	15.5 dia Pier Location	Height on Wall Feet	Ultimate Capacity, Kips
Mean Value	Landside	8	69.9
		13	62.2
		18	54.1
Mean - 1 std	Landside	8	52.3
		13	46.3
		18	40.3
Mean + 1 std	Landside	8	93.6
		13	84
		18	73.3
One Third	Landside	8	64.3
		13	56
		18	49.4



Phi angle	15.5 dia Pier Location	Height on Wall Feet	Ultimate Capacity, Kips
Mean Value	Mid	8	68.5
		13	59.4
		18	50.3
Mean - 1 std	Mid	8	50.7
		13	43.8
		18	36.9
Mean + 1 std	Mid	8	91
		13	79.8
		18	67.4
One Third	Mid	8	62.2
		13	52.7
		18	44



Phi angle	15.5 dia Pier Location	Height on Wall Feet	Ultimate Capacity, Kips
Mean Value	Riverside	8	66.6
		13	55.9
		18	45.5
Mean - 1 std	Riverside	8	49.1
		13	41.1
		18	33.1
Mean + 1 std	Riverside	8	88.2
		13	75.1
		18	61
One Third	Riverside	8	60.1
		13	49.2
		18	39.6

